AMERICAN SEWERAGE PRACTICE

VOLUME III

DISPOSAL OF SEWAGE

AMERICAN SEWERAGE PRACTICE

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BY

METCALF AND EDDY

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AMERICAN SEWERAGE PRACTICE

VOLUME III

DISPOSAL OF SEWAGE

BY

LEONARD METCALF

AND

HARRISON P. EDDY

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PREFACE

The purpose of this volume is twofold: first, to explain in simple, non-technical language the nature of sewage and the changes that take place in it when it is subject to different conditions, and second, to describe the structures designed to produce these various conditions, in order that the character of sewage may be changed to the desired extent before it finds its way into some body of water. The first purpose is carried out in Chapters I to VI inclusive. The remainder of the volume is intended primarily for designing engineers and operators of such plants.

So far as the authors are aware, they have treated the subject in a new way, which has been adopted as a result of their conviction that in the present state of knowledge concerning sewage disposal, highly technical discussions of disputed theories were undesirable in a book intended to be helpful not only to engineers but also to sewer commissioners, lawyers and under-graduate students. Some of the chapters have been rewritten several times in order to avoid repetition and needless technicalities. Other chapters would be materially changed if the practical experience of today but furnished adequate precise data, rather than a certain amount of general information from which have been drawn the inferences here given. This is particularly true of the subjects of screening, the American use of contact beds, the oxidation of sewage directly or indirectly by aeration, and the disposal of sludge.

For the opportunity to present the large amount of information regarding American practice in sewage disposal contained in this volume the authors here gratefully acknowledge the valuable and generous assistance given to them by their engineering colleagues and friends, whose co-operation in this undertaking has been typical of the spirit of mutual professional helpfulness that has been responsible for a large part of the recent progress in this field. In the present state of the subject, the experience in other countries affords much valuable information which has been employed liberally, although this treatise is primarily intended to be a survey of American Sewerage Practice. For a considerable part of this information the authors are indebted to engineers in England, France and Germany, whose cordial aid in their task has proved one of the most pleasant features of the preparation of the volume.

The authors have kept constantly in mind the fact that while there is potential danger to public health in sewage, the disposal of this class of municipal wastes is not alone a technical problem but one which calls

for heavy, continuing expense. It is something in which the sanitarian and the civic economist are as interested as the engineer, and the authors have endeavored to treat the subject in a way to recognize this fact. Skimping funds may often lead to danger to the public health, and the enforcement of requirements for an unnecessarily high degree of purification of sewage to useless waste of money. It is the engineer's duty to safeguard the public health and to advise wise limits of expenditure. This can be done most effectually by insisting that each undertaking shall be considered upon its own conditions and that the trained specialist in this branch of engineering shall be the judge of the significance and applicability of experience gained with disposal works elsewhere. The danger of failure resulting from copying plans of one plant for use in another locality is very real in the field of sewage disposal.

The extent of the sewage treatment works in this country is indicated by statistics compiled under the direction of George M. Wisner, Chief Engineer of the Sanitary District of Chicago. These are based on the census of 1910 and indicate that out of a total population of about 91,600,000 in the United States, about 34,700,000, or 38 per cent., lived in places provided with sewerage systems. Of these systems, those serving a population of 3,900,000, or 11 per cent., were provided with sewage treatment works. The sewage of 89 per cent. of this population in sewered places was discharged untreated into water. This shows the importance of dilution as a means of disposal. About 10 per cent. of the population served by sewers lived in places having basins or tanks for treatment, 3 per cent. where intermittent filters were used, 1 per cent. contact beds, and 4 per cent. trickling filters. Some of these places have two of these methods of treatment in use, so that the total of the figures just given is greater than the 11 per cent. previously mentioned as served by systems with some form of treatment works.

The art of sewage treatment has made radical and important advances during the last 25 years, and it is to be expected that this progress will continue. The number of persons engaged in the study of sewage disposal problems is increasing rapidly, which, with free interchange of ideas, must stimulate more rapid future progress in the perfection of methods. The prospect of such improvements rarely justifies delay in the installation of needed treatment plants, however, for there are now available methods of economically accomplishing any degrees of purification which may be required, nor is the discovery of a better method often cause for just criticism of those responsible for one already in use. Improvements in every field follow careful investigation and change in conditions.

LEONARD METCALF.
HARRISON P. EDDY.

BOSTON, MASS.,
October 6, 1915.

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CHAPTER I

INTRODUCTION: PROGRESSIVE STEPS IN SEWAGE TREATMENT

The tendency to throw refuse into any water course near at hand made disposal by dilution the natural and leading method of getting rid of sewage as soon as sewers were used to carry such material from streets and houses. Cesspools were employed very extensively in the early days of sewerage, which was introduced mainly to remove storm water. This storm run-off was very foul and contaminated the rivers into which it was discharged; this contamination was sometimes increased by the nightsoil gathered by cesspool-cleaning contractors, who threw it into the rivers in many cases. As long ago as 1842, the Poor Law Commissioners of England reported that outfalls should be constructed to remove the sewage of cities to tracts of land where it could be disposed of without polluting the streams. This recommendation fell on deaf ears, for the sewers built for a long time after that date discharged into the rivers. Although a Public Health Act was passed in 1848, it had little effect, and it was not until the Nuisance Removal Act of 1855 was passed, at the close of a severe cholera epidemic, that effective legislation began. The rapid introduction of water-closets about this time rendered tight cesspools, such as were generally used in England, impracticable, and drew attention to the usefulness of sewers for removing house wastes. The evil condition which was fought was understood only vaguely at that time. The legislative purpose was to prevent rivers and other receivers of sewage from becoming offensive to the eye and nose. If neither of these organs could detect anything unpleasant, it was believed that there was no ground for serious complaint against the method of disposal.

Royal Sewage Commission.—In order to find out how to safeguard the rivers most effectively, a Royal Sewage Commission was appointed in 1857 to ascertain the best method of disposing of the sewage of cities. Its final report was not made until 1865. It recommended land treatment of sewage, but expressed doubt as to the profits of sewage farming

under some circumstances. In case a river was polluted with sewage, the town causing the nuisance should be required to stop it, the Commission advised, and where cesspools were endangering health the towns so affected should be required to put in sewerage systems.

Royal Commissions on Rivers Pollution.—The first Royal Commission on Rivers Pollution was appointed in 1865 to ascertain if legislation prohibiting the discharge of sewage into rivers might not result in more serious conditions of other kinds. This body could not agree on a report and was discharged. The second Royal Commission on Rivers Pollution was appointed in 1868 to find out how much restriction should be placed on the discharge of sewage into running water and how the sewage could be utilized and got rid of otherwise than by such discharge into streams. This Commission made six reports. The first was published in 1870. It described various ways in which rivers were polluted and the analytical methods of detecting pollution, and gave the following opinion concerning the three methods of treatment then in use:

"The filtration processes[1] are the best in this respect (the removal of polluting organic matter in suspension, but not that in solution); irrigation ranks next, while chemical processes are somewhat less efficient for the removal of suspended organic matter. But the getting rid of suspended matters is a simple problem compared with the removal of organic matters in solution. It is here that the different processes experience the most severe trial, and it is on the application of this test that the great superiority of downward intermittent filtration and of irrigation over upward filtration and the chemical methods of treatment become strikingly apparent. Thus, in round numbers, it may be said that, on the average, the processes of intermittent downward filtration and irrigation remove from the soluble constituents of sewage (as measured by organic carbon and nitrogen) twice as much polluting matter as that got rid of by the processes of chemical treatment and of upward filtration. Looking only to purity of effluent water, it would be difficult to decide between downward intermittent filtration on the one hand and irrigation on the other, but there are obvious reasons why the latter must, in all but very rare and exceptional instances, be preferred on economical grounds. Intermittent filtration is a costly process with no possibility of any return, while irrigation, although it may in the first instance require a large outlay of capital, affords a hopeful prospect of a return for the capital invested."

This report also dealt with the pollution of the Mersey and Ribble basins, then a cause of great offense. A second report, later in 1870,

[1] "Broad irrigation means the distribution of sewage over a large surface of ordinary agricultural ground, having a maximum growth of vegetation (consistently with due purification) for the amount of sewage supplied. Filtration means the concentration of sewage at short intervals, on an area of specially chosen porous ground, as small as will absorb and cleanse it; not excluding vegetation, but making the produce of secondary importance. The intermittency of application is a *sine qua non*, even in suitably constituted soils, wherever complete success is aimed at." (Royal Commission on Metropolitan Sewage Discharge, 1885).

discussed the A.B.C. chemical process of sewage treatment. The third report, in 1871, was concerned with the Aire and Calder basins, where the nuisance due to dumping sewage and refuse into the water had become serious. The fourth report, in 1872, dealt with Scotch river basins. The fifth report, in 1873, was on river pollution by mining and metal industries. The sixth report dealt with the contamination of domestic water supplies and made the erroneous assertion that dissolved organic matter of sewage origin could be removed from water only to a slight extent by sand filtration. The most influential work of the committee seems to have been the formulation of certain standards of purity for British rivers by forbidding the discharge of various substances into them, which, it was claimed, could be kept out without working hardship on towns or industries.

The quotation, just made, concerning methods of treating sewage is of interest as containing official recognition at that time of filtration as a good method of treatment, although it was then practised only at Ealing and Chorley. The main interest lies, however, in the complete failure to recognize the dilution of sewage as a method of treatment. For many years after the date of the report, dilution was a neglected subject of study except in the case of running water, where it received some attention under the name of self-purification of rivers. Sewage discharged into water undergoes changes comparable in a general way to those which take place when it is passed through earth. If more sewage is added to the water than the latter can transform into inoffensive substances, a nuisance will result; this is equally true if more sewage is discharged on land than it is able to change into similar substances. The latter condition was well known in 1870 and farmland overdosed with sewage was called "sewage sick." While the distribution of sewage over land was then a well-recognized method of sewage treatment, its dilution in water was regarded exclusively as a method of disposal, As a matter of fact, dilution is a valuable method of treatment, and a city which has a neighboring body of water where it can be practised safely possesses an important natural resource.

Land Treatment.—Irrigation of land with sewage was a very old practice when the Second Rivers Pollution Commission made its report. Probably some of the surface drainage of ancient Athens was distributed over meadows near the city. Since the middle of the sixteenth century the sewage of Bunzlau, a German town, had been used for irrigating cultivated ground, and the same thing had been practised in Ashburton and other Devonshire towns from the beginning of the eighteenth century. A famous example of sewage irrigation had existed at Edinburg for nearly two centuries. There the sewage of a part of the city was carried away by Craigentinny brook to the Firth of Forth. Its lower course was through meadow land, and the sewage-laden

water was found to increase greatly the growth of grass on this tract. Gradually the grass lands irrigated in this way were extended until they amounted to about 250 acres. Part of the soil was loam, and part sand and gravel. The irrigated fields were drained and the effluent from the drains was clear and inoffensive. The land was owned by private parties, who used only as much water as they wished. This water was practically weak sewage, and what was not used for irrigation flowed untreated into the Firth. Consequently these meadows were not an example of sewage-treatment works but of irrigation with sewage. The distinction is an important one, because in the first case the treatment of the sewage is the important object and in the second case the raising of crops. The two objects are usually inimical and one must be sacrificed to the other.

Filtration of sewage, by which was meant its passage through earthy material without any attempt to raise crops, was practised at Ealing in 1868, by distributing sewage through beds of burnt clay and coal. This distribution was subsurface and the sewage was not allowed to rise to the top of the beds. At Chorley, fallow land was plowed and sewage turned on it. The Commission was not satisfied with the results at either place, but one of its members, Sir Edward Frankland, was led to conduct a series of experiments which resulted in the favorable opinion of filtration already quoted. He attributed the changes in the sewage as it passed through a filter to chemical causes solely, and it was not until Schloesing and Müntz published in 1877 a report on the influence of micro-organisms on such changes that the bacterial aspect of sewage treatment was pointed out. This was discussed in more detail the following year in a paper by Schloesing and Durand-Claye before the International Congress of Hygiene at Paris.

The first place to adopt filtration according to Frankland's principles was Merthyr Tydvil, where it was selected in 1871 as a temporary expedient on the advice of Bailey Denton. It was followed for 5 months with satisfactory results and was then abandoned in order to use the sewage for irrigation. This engineer constructed many such filters subsequently, usually as auxiliaries to sewage farms, and these were generally successful. Other engineers built filters shortly after the publication of the commission's report, but the principles recommended by Frankland were ignored in their design, and in 1877 it was reported by Robinson and Mellis that the sewage filters constructed by 38 towns were unsatisfactory. In 1880 Baldwin Latham again drew attention to the value of this method of treatment, which he explained as a bacterial rather than chemical process, and 3 years later he designed treatment works using it for Friern Barnet. Filtration on scientific lines has never been extensively employed in Great Britain, because of the scarcity of suitable material for beds. The so-called filters used in

connection with irrigation farms are, in many cases, merely underdrained fields of somewhat porous soil not well adapted for the use to which they are put.

Chemical Precipitation.—Chemical precipitation was also a well-established method of sewage treatment in Great Britain in 1870, at the time of the Second Rivers Pollution Commission's report. Many patented processes had been devised, beginning with one proposed in 1762 by de Boisseau. The partial clarification of sewage and trade wastes by plain sedimentation had been practised for many years along the most polluted streams in order to reduce the nuisance which existed in their neighborhood. The results were often unsatisfactory, doubtless due in some cases to neglect to remove the sludge, which gradually occupied so much room that the sewage was compelled to pass quickly through the tanks and little opportunity for sedimentation was afforded. The addition of chemicals to the sewage to increase the degree of clarification was manifestly worth trying. Lime was used as a precipitant in most cases, sometimes alone, but more often in combination with chloride of lime, chloride of magnesium, sulphate of alumina, phosphate of alumina, green copperas, black-ash waste, charcoal, herring brine, or some other one of many substances. Occasionally several substances were used in order to obtain special results. This was the case with the A.B.C. process, which was strongly advocated because its sludge would have unusual merits as a fertilizer, due to the use in it of alum, blood, charcoal and clay. Ferrozone was brought out as a precipitant and was used somewhat. The effluent from sedimentation aided by ferrozone was sometimes filtered through a patented material called "polarite." An artificial material called alumino-ferric, of low price, gradually won its way into considerable favor in England on account of the ease with which it can be used in treating sewage of fluctuating quantity.

Seine Pollution Commission.—English opinions at this date were duplicated in France, where the Seine Pollution Commission reported in 1874 as follows:

"To prevent the pollution of the Seine by the waters of the intercepting sewers, the most economical, practical and efficacious means consists in using them in the irrigation of a sufficiently permeable soil. Different kinds of tillage (above all, that of market gardens) will find in these waters the moisture and manure necessary for them. The experiments made in the plain of Gennevilliers are entirely conclusive in showing, not only the luxurious vegetation which may be produced by irrigation, but its harmlessness in respect to health, as well as the perfect purification of the sewage, which returns to the river after having traversed a subsoil naturally permeable or sufficiently drained. As to purification by chemical processes, and in particular by sulphate of alumina, the commission is of the opinion that it will not constitute a complete and practical solution of the problem."

Local Government Board.—The Local Government Board of England was constituted on August 14, 1871, on the recommendation of a royal sanitary committee appointed in 1869 to investigate the operation of the public health laws and to suggest improvements in them and their administration. No money can be borrowed for sewage treatment works in England unless the plans have received the sanction of the Board or the city has been authorized by a special act of Parliament to build the works. In 1875, before adopting any general policy respecting the approval of different methods of treatment, the Board appointed a committee to investigate sewage farming, irrigation with sewage, and sedimentation and chemical precipitation, and report on the nature, cost, offensiveness (if any), operating and maintenance expenses, and efficiency of treatment of at least four examples of each type. Sir Robert Rawlinson, the highly gifted and much respected chief engineering inspector of the Board, was a member of this committee, which reported as follows concerning methods of treatment:

"As far as we have been able to ascertain, none of the existing modes of treating town sewage by deposition and by chemicals in tanks, appears to effect much change beyond the separation of the solids and the clarification of the liquid. The treatment of sewage in this manner, however, effects a considerable improvement, and, when carried to its greatest perfection, may in some cases be accepted.

"Town sewage can best and most cheaply be disposed of and purified by the process of land irrigation for agricultural purposes, where local conditions are favorable to its application, but the chemical value of sewage is greatly reduced to the farmer by the fact that it must be disposed of day by day throughout the entire year, and that its volume is generally greatest when it is of the least service to the land.

"Land irrigation is not practicable in all cases, and, therefore, other modes of dealing with sewage must be allowed.

"Towns situated on the seacoast, or on tidal estuaries, may be allowed to turn sewage into the sea or estuary below the line of low water, providing no nuisance is caused, and such mode of getting rid of sewage may be allowed and justified on the score of economy."

Although the Board has refused to adopt any general rules to govern the design of treatment plants, it has been governed by the recommendations quoted, and has rarely sanctioned any plans which did not provide land treatment to furnish the final effluent. This practice exercised a strong influence on the design of treatment works in Great Britain, and accounts for the many attempts to use land for irrigation or filtration which would be regarded in other countries as practically useless for the purpose.

It is necessary to keep in mind that the streams of England are so small that the amount of sewage and wastes which may be added

to them without offense is also small. Industries flourished on the banks of many of these little streams along which highly offensive conditions existed in many places. Solid wastes were dumped on the banks at some points to such an extent that the cross-section of the water channel was much reduced. Manufacturers protested that the water of the streams was rendered unfit for industrial purposes by the municipal sewage discharged into them, but they saw no objection to their own practice of throwing into the rivers the solid and liquid wastes from tanneries, dye works, paper mills and wool-scouring works. In many cities, the streams were regularly used as public dumps for ashes and cinders, the refuse from torn-down buildings was thrown there and any other material which it was desired to remove from sight as cheaply as possible. Dead animals were cast into the water; a lock at Manchester once had 19 dead dogs in it at one time. Children sometimes amused themselves by lighting the gases which bubbled from the foul waters, which at one place were officially described as "a boiling, stinking mass."

British Legislation Affecting Sewage Disposal.—In 1865, a Sewage Utilization Act was passed which permitted local authorities to combine to protect water courses from pollution by sewage. In 1866 it was amended by giving power to a central body to compel local authorities to construct and maintain sewers. In 1866, 1868, 1869 and 1870, there was legislation to improve the methods of dealing with sewerage problems. In 1872, an act was passed which facilitated the combination of local authorities for joint sewerage works and divided the country into urban and rural sanitary districts. Finally, in 1875, all these acts were replaced by the Public Health Act, which is the present general law for sewage disposal in England and Wales, but must be interpreted in connection with the River Pollution Prevention Acts of 1876 and 1893. The latter make it an offense for any person to discharge or permit to be discharged into any stream any solid or liquid sewage matter. The Local Government Act of 1888 specifically authorized country councils to enforce the River Pollution Act of 1876, and the extremely offensive conditions in certain river basins were met by acts forming local boards to deal summarily with these problems. The Mersey and Irwell Board and the West Riding of Yorkshire Rivers Board were organized under such special legislation, and have dealt extensively with the treatment of industrial wastes as well as house sewage.

In all this legislation relating to river pollution, the discharge of solids into the streams received a degree of attention not paid to it elsewhere, as a rule, because the English practice of using the streams as public dumps for every class of refuse had produced exceptional conditions in the small water-courses of that country. In short, the removal and

treatment of sewage in suitable manner,[1] was considered as important for preventing or reducing offensive conditions in rivers as for protecting public health.

Massachusetts State Board of Health.—The next important step forward in sewage treatment was taken by the Massachusetts State Board of Health. In 1872, it was ordered to investigate the subject of sewerage, including sewage disposal and stream pollution. This work was done by Prof. William Ripley Nichols and Dr. George Derby, who reported that the water-courses were in danger of contamination and even, in places, of the pollution like that causing so much nuisance at the time in England. In 1875 the Board had James P. Kirkwood make an examination of the contamination of Massachusetts streams, who reported that only nominal attempts had been made to prevent river pollution, because the machinery for doing this was cumbersome, expensive and slow in producing results. The contamination was not the same in the different river basins, and in the same basin, in some cases, there were evidences of self-purification, by which the effect of sewage discharged at one point gradually became less marked as the water passed downstream and finally disappeared. With this report was one on the disposal of sewage, written by Dr. C. F. Folsom, which exerted much influence by explaining what was being done in England and Europe to abate the nuisances due to improper methods of treating and disposing of sewage. The Board reported:

"The principle should be established that each community should dispose of its own filth without allowing it to be a source of offense to others. . . In inland cities and towns, irrigation would be likely to be successful, and not involve a large annual cost."

At the time this was written, sewage treatment was practised in this country at Augusta, Me., where the State Insane Asylum irrigated hay fields and vegetable gardens. A little later the Worcester Insane Asylum began sewage farming, and both surface and subsurface irrigation became well known soon. The treatment of the sewage of the city of

[1] It was pointed out in the Introduction to Volume I of this treatise that the main drainage works of London were undertaken largely on account of the foul condition of the River Thames at the metropolis. These works delivered the sewage from the district north of the river to Barking, and from the district south of the river to Crossness. These places are 11 and 13 miles respectively below London Bridge. The works were completed in 1864. The sewage was stored in large basins from which it was discharged at high tide into the river. The river conditions were improved for some years. Complaints of unsatisfactory conditions at Barking and Crossness and below these points finally increased in numbers, however, until (1882) it seemed desirable to appoint a Royal Commission on Metropolitan Sewage Discharge to investigate the subject. In a report made in 1884 the adoption of some process of deposition or precipitation was recommended to clarify the sewage prior to its discharge. Investigations were made by W. J. Dibdin which led him to recommend precipitation with lime and protosulphate of iron. The works were reconstructed in accordance with those suggestions, and precipitation was begun in 1889 at Barking and 1891 at Crossness. In 1911 these two works treated an average of 386,500,000 U. S. gal. of sewage daily, from a district having 5,336,100 population.

Worcester, Mass., aroused protracted controversy. The scientific advisers of the State Board of Health recommended the city to employ filtration and later testified against the city in hearings on a bill to force it to purify its sewage before discharging it into the Blackstone River. The bill was not passed, but in 1883 the city directed its engineer, Charles A. Allen, to investigate methods of treatment and report on the subject. In 1886 the legislature directed the city to begin to treat its sewage not later than 4 years from the passage of the act, before discharging it into the river. Thereupon the city directed its engineer to prepare plans for sewage treatment works as soon as possible, and he recommended chemical precipitation as better suited for the conditions during winter than irrigation or filtration, as less expensive, as returning to the Blackstone River the largest possible amount of effluent, as best suited to the unusual composition of the Worcester sewage, and as capable of subsequent extension and operation with land treatment as experience showed what was best. The plant was put in operation on July 2, 1890, being the second using chemical precipitation to go into service in the United States. The first was built about a year earlier at East Orange, N. J., from the plans of Carroll Phillips Bassett.

Lawrence Experiment Station.—"In 1888 the Massachusetts State Board of Health was directed by the legislature to assume "the general oversight and care of all inland waters." Authority was given to employ necessary technical assistants, to make examinations of these waters, and "to conduct experiments to determine the best practicable methods of purification of drainage and sewage or disposal of the same." The advice of the board was made a necessary precedent to the granting of legislative authority for the execution of any plans for water supply, drainage or sewerage. The Board determined that the available information concerning methods of sewage disposal was inadequate as a basis for giving official advice to cities, towns and individuals, and the Lawrence Experiment Station was accordingly established under the direction of the engineer member of the Board, Hiram F. Mills, Hon. M. Am. Soc. C. E. It at once began a series of investigations of the nature of filtration which has continued to the present time and has undoubtedly been the leading source of inspiration and information since the publication of the first report on its research work in 1890. The existence of the Worcester treatment plant also lead to an investigation of the nature and results of chemical precipitation, reviewed in the same report, which was of much influence in checking a tendency of that time toward overestimating the efficiency of this process as respects producing an effluent subject to no offensive changes.

The great value of the 1890 report was its clear demonstration of the biological nature of filtration. This was not an original discovery at

Lawrence, but the elaborate experimental work done there threw much-needed light on leading phases of the subject. The filters were made of material ranging in character and size from peat, loam and very fine sand to large gravel, and the sewage was applied intermittently and continuously, and from below as well as above the surface. The results were reported in great detail in each annual report, and the information concerning the action of bacteria proved an inspiration to many investigators and resulted at once in an improvement in the design of intermittent filters in the United States, and in attempts in England to produce equivalent bacterial action in beds composed of materials other than sand.

Nitrifying Trays.—Pasteur discovered that some bacteria, which he called aerobic could exercise their functions only when air was present in the sewage; that others, called anaerobic, could operate only where all oxygen was absent, and that a third class, called facultative, could operate under either aerobic or anaerobic conditions, although not always with equal strength. The aerobic and facultative bacteria are of many kinds and accomplish different results. One of the early attempts to utilize bacterial action on sewage in an intensive way was made in nitrifying trays. In an experimental plant at Ashtead, England, W. D. Scott-Moncrieff used 9 of these trays, one over the other, 3 in. apart. Each tray was perforated and contained 7 in. of coke 1 in. in size. The sewage was discharged into the top tray by tipping troughs, and then percolated through the coke and dropped down from tray to tray, about 10 minutes being occupied in this passage.

It was held by the inventor that this percolation and aeration in successive stages probably developed in each tray the species of bacteria best adapted for the stage of the purification process normally being conducted in that tray. This view was confirmed by the deterioration in the quality of the effluent when the trays were transposed, followed in about 2 days by recovery to a satisfactory condition. Although interesting for several reasons, these trays were never used to any considerable extent, and the main attempts to accelerate the desired changes in sewage were along lines resulting in what are known as contact beds and trickling or percolating filters.

Contact Beds.—Sir Alexander Binnie, Chief Eng. of the London County Council, was dissatisfied with the results of chemical precipitation at Crossness and Barking, owing to the large amount of sludge produced, and on his advice the Main Drainage Committee of the Council ordered, in 1891, a series of experiments to be made at Barking on the lines of the Massachusetts researches. Instead of dosing the beds as had been done at the Lawrence intermittent filters, sewage was applied at high rates of flow for 8 hours continually, and to prevent its passage too rapidly through the bed, the outlet was trapped, so that the bed remained

full of sewage, and the effluent flowed away only as fast as the influent entered the beds. After running in this way for 8 hours, the filter was drained and allowed to stand empty for 16 hours. These experiments were under the immediate charge of W. Santo Crimp and W. J. Dibdin, who found that the removal of dissolved organic matter was less, although the clarification was greater, with sand and gravel than with coarser material. A 1-acre bed was then constructed consisting of 3 ft. of coke breeze and 3 in. of gravel. At first this was filled and emptied twice daily, but later the filling took 2 hours, then the bed stood full for an hour, and finally it was drained in 5 hours. It rested empty from 10 p.m. on Saturday to 6 a.m. Monday.

In 1894, the same treatment was undertaken at Sutton, using both coke breeze and burnt clay for the beds. The population of the town was about 13,000 at that time. For nearly 3 years these beds were worked with an effluent which had been strained, treated with chemicals and allowed a period of sedimentation. In November, 1896, this process was changed to screening and passage through a bed of coarse burnt ballast called a "bacteria tank." Before this bed was put in operation it was dosed with liquid having bacteria which were believed to be helpful in producing the desired changes in sewage. The effluent from this bed was then applied to the contact beds as before. This plant remained in service, and received a great deal of attention. The results were considered satisfactory, but success was dependent upon careful supervision of the operations and the analyses showed that a large proportion of the total degree of purification was due to the changes that occurred in the bacteria tank. The authorities of Oswestry adopted the system in 1898, but in a somewhat modified form. The bacteria tank used at Sutton was replaced at Oswestry by a coarse primary contact bed and its effluent was delivered to a finer secondary contact bed.[1]

The most extensive use of contact beds has been made at Manchester, England. After trying filtration and chemical precipitation without satisfactory results, an investigation of sewage treatment methods elsewhere in England was begun in 1897. In the following year experimental contact beds were constructed at Manchester and operated under the supervision of Baldwin Latham, Percy F. Frankland and W. H. Perkin, Jr. and this experimental work was increased in scope later. It has been continued ever since and the records of it in the annual reports of the Rivers Department of the city contain much valuable

[1] Among the first contact beds constructed in the United States were the following: a small plant for the Glenview Golf Club near Chicago (1898–1899, Alvord & Shields); Depew, N. Y. (1901, City Wastes Disposal Co.); Glencoe, Ill. (1901, Cameron Septic Tank Co.); Mansfield, O. (1902, Snow & Barbour); Plainfield, N. J. (1902, J. O. Osgood); Fond du Lac, Wis. (1902, Geo. S. Pierson); Clayton, Mo. (1902, Cameron Septic Tank Co.) A list of the sewage treatment plants in the United States at the beginning of 1902 was published in *Engineering News*, April 3, 1902.

information. One advantage in the use of contact beds at Manchester is that the small available operating head is enough for this method of treatment but insufficient for trickling filters. In 1914 the works included 10.1 acres of storm water, grit and open septic tanks, 46 acres of primary contact beds, 34 acres of secondary contact beds, 5 acres of secondary beds under construction and 26.8 acres of filters used for storm water, in addition to 99.6 acres of roads, wharves, railways, embankments, buildings, sludge tanks and similar property. The average amount of sewage received at the works was 40,676,000 U. S. gal. a day, of which 97.2 per cent. was treated by the contact beds, although not all of it by double contact, and 2.8 per cent. was passed through sedimentation basins and then discharged without further treatment into the Manchester Ship Canal. The efficiency of the treatment at these works increased as the operators gained experience, but the results have not been satisfactory. "While the possibilities of nuisance arising from the canal are not increased by the discharge of the present effluent," the 1914 report states, "the average effluent leaving the works does not meet the requirements of the Mersey and Irwell Joint Committee nor can it be considered satisfactory."

Slate Beds.—The slate bed, so-called, was developed by Dibdin as a result of his observation of the working of the Sutton contact beds. He found that they did not work satisfactorily when filled three times a day and he accordingly sought some method of reducing the quantity of sludge stored in the beds, which diminished their capacity. He considered that bacterial action in the presence of air was desirable for this purpose and eventually selected a bed filled with horizontal slate plates separated 1 to 4 in. as the best apparatus for this purpose. This was first employed at Devizes about 1904, and was reported by the designer about 2 years later (*Proc. Inst. C. E.*, vol. clxiv, part ii) as doing double as much work as a new coke bed and proving to be easily cleaned by flushing with water. The sludge was put on land, where it rapidly underwent disintegration, and was stated to be composed largely of living bacteria, "with indigestible matter and a certain quantity of mineral matter."

Septic Tanks.—One of the leading contributions of France to sewage disposal was the liquefaction of suspended organic matter in sewage which was without oxygen and contained in a closed receptacle. In December, 1881, and January, 1882, there appeared in *Cosmos*, a French journal, a description of an air-tight chamber or vault, called Mouras' Automatic Scavenger, in which the solid matters in raw sewage were dissolved. In 1882, the Institution of Civil Engineers described (*Proceedings*, vol. lxviii, page 502) an installation of this type receiving the sewage of about 150 persons, and it is believed that many installations were made, particularly in Paris. The interesting feature of this

tank was that both the inlet and outlet pipes were sealed, so that the interior was practically air-tight. This was different from the water-tight cesspools which had been previously used for many years, for little or no attention was paid to trapping their inlets or outlets. In many cases the liquid contents did not overflow through a pipe, but had to be pumped out, and it was a matter of common knowledge among those who had to care for such tanks that a part of the solids entering them must have been liquefied during the stay of the sewage in them.

An example of a sewage tank having a trapped inlet and outlet is afforded by the two-chamber basin constructed in 1876 at the State Insane Asylum at Worcester, Mass., in connection with a sewage irrigation system. This method of construction was advocated by E. S. Philbrick, who had much influence on sanitary matters at that time, and was used in the tanks built in 1882 at Lawrenceville, N. J., for a private school, and in 1883 at Concord, Mass., in the tanks for a reformatory.

Experimental and practical work with the liquefaction of the solids in sewage without oxygen was taken up by W. D. Scott-Moncrieff about 1890, as a result of his observations on the changes in the organic matter in sewage flowing long distances. In 1891 he constructed at Ashtead, England, a tank with an empty space of 5 cu. ft. at the bottom under a grating supporting a bed of stones. The sewage of a household of 10 persons was admitted to the tank below the grating and passed upward through the spaces between the stones. At the end of 7 years, the sludge which remained below the grating was cleared out and was readily disposed of on 9 sq. yd. of land. This process was investigated in 1892 and again in 1893 by Dr. A. C. Houston, and his report of the latter date contained very favorable opinions concerning the value of this method of treating sewage as a preliminary to filtration or irrigation and also concerning its apparent great decrease in the volume of sludge to be handled. The cost and difficulty of satisfactory disposal of sludge from sedimentation tanks and precipitation basins were so great that methods of reducing one or both of them were eagerly sought, and the liquefaction of sewage attained in this Scott-Moncrieff "anaerobic tank" immediately attracted much attention.[1]

Unfortunately the effluent from such a tank was found to be subject to undesirable changes and consequently about 1895 there arose an opinion that there should be two stages in the treatment of sewage, the first stage to be one carried out in the dark and with the oxygen reduced to a minimum and the second one with abundant oxygen in the sewage. In the first stage it was sought to liquefy all the solid organic matters, and in the second stage to change the effluent from the first so that it would cause no offensive conditions at the place of final disposal.

[1] The inventor was granted an American patent, No. 624985, for a treatment process involving the anaerobic tank and the nitrifying trays mentioned on page 10.

When this method of treatment had reached this stage, in 1895, City Surveyor Donald Cameron of Exeter constructed what he called a "septic tank" for the preliminary treatment of screened combined sewage averaging about 60,000 U. S. gal. per day.[1]

The tank was 65 ft. long, closed, and held about 64,500 U. S. gal. There were no baffles or other obstructions. Both inlet and outlet were trapped. There was an inspection chamber with glass sides through which the interior conditions of the tank could be observed whenever desired. Here it was found that under deoxygenated conditions, a scum 2 to 6 in. thick formed on the top of the liquid. Below it the sewage was clear for a considerable part of its depth, but bubbles of gas were rising uninterruptedly through it from the sludge at the bottom. It was believed by some investigators that a large part of the mineral matter in the sludge was raised in some mysterious way by the gas bubbles and was carried off in the effluent, but this opinion was found to be unjustified by the facts.

The effluent from this tank was discharged over a weir to aerate it and then delivered to contact beds by automatic apparatus arranged to give each bed its proper time of filling, standing full, discharging and standing empty. The results were such that in 1897 the Local Government Board approved the treatment of the sewage of the entire city of Exeter by this means. The early opinion that the septic tank did away with most of the expense involved in disposing of sludge led to great interest in it in the United States as well as in England, particularly as the experience gained with tanks designed by Prof. A. N. Talbot was held by many engineers to warrant this belief. In 1894, he built at Urbana, Ill., a covered tank with a submerged outlet, in the expectation that in it there would be some of the liquefaction which occurred in the Mouras apparatus. This proved to be the case and in the following year he designed a larger tank for Champaign, Ill., which was built in 1897. It was about 37 ft. long, 16 ft. wide and had a wet depth of about 5 ft. It was divided by a longitudinal center wall into

[1] The report of the Massachusetts State Board of Health for 1894 stated that storing fresh Lawrence sewage for 24 hours doubled the free ammonia and decreased the organic nitrogen present one-half. Other changes which took place in sewage during storage were described, including a considerable increase in bacteria. So far as septic action is concerned, therefore, it was probably described in print before it received publicity from Cameron and the distinctive name now applied to it. In the Board's report for 1908, the difference between settling and septic tanks is stated on page 483 as follows: "The essential difference between settling tanks and septic tanks is that the solid matters deposited in the former are removed at frequent intervals and otherwise disposed of, while with the latter the sludge is allowed to remain for longer periods in the tank, where it is subjected to hydrolytic or bacteriolytic action. By these means a portion of the organic matter is converted into unoffensive gases or into soluble compounds which pass off with the out-flowing sewage. . . . . . As sludge destruction is dependent on slow bacterial action, and as that action may not become operative immediately, it is essential, to get the best results, that septic tanks be cleaned only when absolutely necessary."

two basins each of which had three vertical baffles reaching 2 to 3 ft. below the surface of the sewage. The flow of sewage was about 300,000 to 1,000,000 gal. a day. The tank was covered by a brick building with a shingle roof, the doors and windows being closed tightly. The operation of the tank was described by Talbot in *Engineering News*, August 17, 1899, and that article had a decided influence on the construction of septic tanks in the United States. In particular it did away with the restriction of the term "septic tank" to closely covered basins, and was largely responsible for its extension to open tanks from which the sludge was removed only at intervals of some months.[1]

The British practice was to retain the sewage in these tanks about 24 hours, while the longest period of detention in the Champaign tank was less than 2 hours. This difference was pointed out editorially in the journal containing Talbot's paper, and several American engineers, notably Alvord & Shields, designed plants which permitted a wide range in the time the sewage was detained in the septic tanks.[2] The information obtained from the early plants was not very satisfactory because of the manner in which most of them were operated, amounting practically to neglect, but it soon became apparent that sewage could be held too long in septic tanks, because prolonged anaerobic action made the subsequent filtration less effective. This was an important early contribution to the practical operation of such tanks, which was overlooked because of the sudden check put upon the construction of septic tanks by the owners of the American patents.

Septic Tank Patents.—There were a number of patents taken out in the United States by Cameron and his associates, but only one was on the process of septicization, the others being for apparatus. The process patent was No. 634,423, and was vigorously upheld by its owners, who notified cities contemplating the use of septic tanks that any such use except under a license would be made a subject of litigation. Some municipalities took out licenses, many instructed their engineers to give up any projects likely to lead to suits, and a few ignored the

[1] In reporting on their experiments at Manchester, England, Latham, Frankland and Perkin stated that there was no difference between the effluents of open and closed septic tanks, and this influenced *Engineering News* to advocate in 1899 the omission of covers for such tanks. This opinion was confirmed by experience in the United States, while in England the Royal Commission on Sewage Disposal stated in 1908: "As regards digestion of sludge and quality of tank liquor, a closed tank possesses no advantages over an open tank. There is less risk of nuisance if the tank and the feed channels to the filters are covered in."

[2] "The writer has found that septic tanks are not to be designed on haphazard principles, and has developed a theory from 4 years' practical experience in the operation of such tanks, that the particles of every sewage require a rest or fermentation period within the tank the length of time of which must be adapted to their temperature, their concentration, their character and the volume of flow. . . . . . . In some of our more recent tanks 5 compartments of varying capacity have been introduced, which, when worked singly or in combination, allow almost any considerable fraction of the whole capacity to be utilized for the time being." John W. Alvord, *Jour. West. Soc. Engrs.*, April, 1902.

possibilities of trouble in the courts. Among the last was Saratoga Springs, N. Y., where treatment works including septic tanks were built from the plans of Snow & Barbour. These works were described in detail by F. A. Barbour in *Jour. Assoc. Eng. Socs.*, vol. xxxiv, page 33 (Feb., 1905), and presented the most complete application of anaerobic liquefaction made in this country up to that time. There are 4 septic tanks, each 91.5 × 51.5 ft. with a sewage depth of 7.75 ft. at the inlet end and 8.25 ft. at the outlet end. They have groined roofs and the inlets and outlets are trapped. In the early days of their operation they furnished an effluent entirely free from dissolved oxygen, which was aerated before it was discharged over the filter beds provided to complete the treatment process.

The owners of the American process patent of Cameron brought suit for its infringement against the village of Saratoga Springs which was decided against them in the trial court. This decision was reversed on appeal, 151 Fed. Rep. 242. In the latter decision, the court ruled that while the liquefaction of solids had undoubtedly taken place in older types of tanks for sewage treatment, "Cameron was absolutely the first to instruct the art that the problem of removing sludge could be practically eliminated (irrespective of securing other advantages), by providing the anaerobes with a workshop in which they might act upon the solid contents of the flowing current, unhampered by the presence of air, oxygen, agitation or aerobes." The court laid particular stress upon the liquefaction of solids in the septic tank and the absence of dissolved oxygen in the effluent.

Owing largely to the very strong opposition to the payment of licenses for the use of the patent, which were considered far too costly, organizations to contest the patent were formed. At the same time engineers devised methods of operating the tanks so as to leave a little dissolved oxygen in the effluent and planned to remove the sludge at intervals which would, in their opinion, not bring the tanks under the definition of sludge-less basins which formed a leading feature of the decision of the Circuit Court of Appeals. These methods of avoiding infringement were never tested in the courts, as the owners of the patent were occupied in litigation concerning its life. The Septic Process League formed to contest the patent claimed that it expired on Nov. 8, 1909, the expiration of the life of the British patent, whereas the owners contended this was not the case. Finally in a suit for infringement at Knoxville, Iowa, which was appealed to the U. S. Supreme Court, both parties agreed to present the following statement of fact to the court:

"That the allegation in the Plea to the effect that the invention patented in the United States Letters Patent, issued to Donald Cameron et als. on a process of and apparatus for treating sewage, No. 634423, dated Oct. 3, 1899, set forth in said bill, had been previously patented in a foreign

country by said patentee, to wit: in the United Kingdom of Great Britain, by Letters Patent, dated Nov. 8, 1895, and that the said last-mentioned patent of Great Britain expired on Nov. 8, 1909, being the expiration of the term for which it was granted, may be taken as true."

The Supreme Court decided on Jan. 20, 1913, that the American patent expired in 1909. The effect of this long litigation was to prevent in the United States the development of such a favorable opinion of septic tanks as existed in Great Britain. This was inevitable when conservative engineers hesitated to involve municipalities in suits in the federal courts and the engineers who were not so conservative were frequently restrained from risking such suits by the legal advisers of the cities. A large number of tanks were probably built under the name of septic tanks from the plans of persons without adequate knowledge of the principles of sewage disposal, and the failure of these to accomplish the anticipated results and the nuisance caused by some of the worst of them, did not help raise the status of this method of treatment. The advent of the two-story tank, described a little later, was welcomed by American engineers with feelings of relief that cannot be understood without a knowledge of the bitter opposition to the Cameron process patent.

Two types of the new two-story tanks are also patented, but the license fee charged up to date for the only one used in the United States has been small and the patentee, Dr. Karl Imhoff, has given much valuable advice without fees to the engineers of the licensees. As a result the Imhoff tank rapidly became established firmly in the United States, in spite of the strong aversion of the civil engineering profession to the use of patented processes.

Trickling Filters.—The early experiments of the Massachusetts State Board of Health at Lawrence proved that "the slow movement of the sewage in thin films over the surface of the stones, with air in contact, caused a removal for some months of 97 per cent. of the organic nitrogenous matter as well as 99 per cent. of the bacteria." This demonstration of the value of coarse material for filters was followed in 1891 by the operation of a gravel filter at a rate of about 200,000 gal. per acre daily, the sewage being applied in 60 to 70 doses a day. The coarse beds operated in this way, now called indiscriminately trickling filters, sprinkling filters and percolating filters, attracted more attention in England than in the United States, because of the extremely high cost of sand filters in most parts of the kingdom and the abundance of material from which coarse filters could be made. The field of their active development was, therefore, transferred to England.

One of the earliest filters of this type was constructed at Salford, England, by Joseph Corbett about 1893, the inspiration for the design being furnished by the reports of the Lawrence experiments. His experiments lasted many years (*Engineering News*, Feb. 26, 1903), and in

the course of them he tried a variety of methods of applying the sewage to the filters, finally adopting jets which sprayed the liquid into the air. Other investigators, particularly Col. Geo. E. Waring at Newport, R. I., and Sidney R. Lowcock at Malvern and Wolverhampton, England, employed a top layer of fine material to effect an even distribution of sewage over the main mass of coarse material, and forced air into the latter in order to have it thoroughly aerated. One of the earliest investigators was F. Wallis Stoddart of Bristol, England, who has stated that as early as 1883 he publicly demonstrated the possibility of operating coarse filters so as to produce the desired changes in sewage. His first demonstration on a working scale seems to have been made in 1898, however, the liquid being distributed as a rain or spray from troughs. These were corrugated sheets, with perforations along the ridges and small projections below the valleys. The liquid passed through the openings and down the bottom slopes to the projections, from which it dropped.

Corbett experimented in 1894 with revolving arms to distribute the sewage. Four years later this method of distribution was installed by Whittaker & Bryant at Accrington, and it was used experimentally about the same time by the Candy engineering firm at Reigate. In these devices the rotating arms were perforated pipes, and some trouble was experienced by clogging of the holes. Mather & Platt were among the first to use revolving open troughs. The Fiddian distributor, brought out somewhat later by Birch, Killon & Co., of Manchester, was essentially a long overshot water-wheel, pivoted at the center of a circular filter bed and carried at its outer end on a truck running on a track around the circumference of the bed. The sewage was admitted to the buckets near the top of this elongated water-wheel and its weight caused the arm to revolve, the sewage falling from the buckets upon the broken stone as the arm moved. In these rotating devices there was a tendency for too much sewage to fall upon the stone in the center of the bed, and, moreover, circular beds could not be used satisfactorily for large treatment plants, owing to the amount of ground space not usefully occupied. Accordingly the Fiddian distributor was modified so as to move automatically from end to end of rectangular beds, and other devices were also brought out for the same purpose.

One of the earliest trickling filters in the United States was constructed at Madison, Wis., in 1901, from the plans of Profs. J. B. Johnson and F. E. Turneaure. It was not strictly a sprinkling filter, as the liquid was distributed below the surface of the bed in order to prevent freezing. The earliest filters of the trickling type at the Lawrence Experiment Station were dosed on the surface and the statements regarding their satisfactory operation, published in the reports of the Massachusetts State Board of Health for 1901, 1902 and

1903, convinced many engineers that such filters would prove useful in those parts of the country where freezing weather would not interfere with the distribution of the sewage. These reports indicated a preference for trickling filters over contact beds for such sewage as was received at the Experiment Station.

In 1903, Dr. Rudolph Hering recommended trickling filters for Atlanta Ga., and about the same time Columbus, Ohio, on the advice of Hering & Fuller, undertook a detailed study of methods of treating the sewage of that place in order to prevent nuisance when it was discharged into the Scioto River. In his report on the Columbus investigations, George A. Johnson stated in 1905 that trickling filters, with the sewage sprayed upon them from nozzles, had been operated successfully through a severe winter. As apprehension of interrupted service during the winter had been the chief objection to trickling filters in the United States, this report resulted in their rapid introduction, the largest installation for a number of years being that at Columbus, designed by John H. Gregory. Other early plants were constructed at Washington, Pa., the agricultural college of the University of Minnesota, and Mt. Vernon, N. Y. The type was also adopted by Calvin W. Hendrick about the same time for the large treatment works at Baltimore, but the magnitude of that plant did not permit the completion of the filters until some years later. The recommendation by Hering to use these filters at Atlanta was not adopted for several years, but eventually they were built. This type of filter quickly gained favor after the success of the early installations was known and in some places, as at Gloversville, N. Y., and Aberdeen, S. Dak., where the winter conditions are severe, roofs have been provided to protect the beds.

Mention should also be made here of the Hamburg type of trickling filter worked out by Dr. W. P. Dunbar, in 1901, for institutional plants, and used the next year in works at Unna for treating the sewage of a population of 10,000 persons. The type has since been used at a number of places in Germany. The essential characteristic is the use of fine material on the surface to distribute the sewage over the coarse material below. Dunbar recommends using about 20 in. of 0.04 to 0.12-in. material on top, then 4 in. of 0.12 to 0.4-in. material and then 4 in. of 0.4 to 1.2-in. pieces over the coarse filter. Clogging of the surface layer can be prevented, he states ("Principles of Sewage Treatment," page 221) by occasionally turning over the top 4 to 6 in. with a shovel and allowing the filter to rest for a day or two.

As the effluent from trickling filters contains considerable suspended matter, it is generally passed into settling tanks before its discharge into small streams which it is desired to keep free from any appearance of contamination. The sludge from these tanks is not so offensive as that from basins in which raw sewage has deposited some of its suspended

matters and under the most favorable conditions may have an odor resembling that of garden mold.[1]

Two-stage Tanks.—From about 1895 to 1905 there was a general development of the opinion that by means of bacterial action most of the organic matter, dissolved and undissolved, could be reduced by treatment of various kinds, singly or in combination, to a condition meeting every requirement between mere clarification and complete purification. The septic tank, sedimentation basin, and coarse contact bed were often spoken of as entirely satisfactory means of preparing sewage for final finishing by intermittent filters or fine contact beds. But while these views were being spread broadcast by technical papers, the men in charge of British treatment plants began to question their accuracy. Among these managers was T. Hughes, in charge of triple contact filters at Hampton, England. Here the anticipated liquefaction of organic solids by bacteria did not take place, the beds became clogged with sludge, and the cost of keeping up the efficiency of the plant was so great that the manager made the comment, formerly widely circulated and frequently quoted among sewage works operators, that the best organism he had at the plant was a man with a barrow.

Dr. William Owen Travis, the local health officer, was in charge of the operation of the Hampton sewage disposal plant, and he sought the cause of the troubles at the contact beds with a mind unhampered by engineering precedent. His study[2] convinced him that sewage contained a large amount of non-settling solids, much of it so mascerated and reduced that it was almost in a state of solution. This class of matter he considered to be largely responsible for the clogging of the contact beds, and he proposed to remove it by utilizing a characteristic of it which was pointed out in 1861 in a paper by Thomas Graham in the *Philosophical Transactions* (vol. cli, page 183) of the Royal Society. This characteristic is a tendency for this fine, thoroughly mascerated matter to collect on the surfaces with which it comes into contact, particularly on porous surfaces. In 1904 Travis put in operation a model consisting of a settling tank, with an inlet at the sewage level and an outlet about half way from the surface to the bottom of the sewage, and

[1] The development of a better knowledge of sedimentation by German investigators belongs chronologically at this place, and the same is true of fine screening. It is desirable, however, to defer mention of them until a later page, in order to explain at the same time the conditions in Germany which turned attention toward other directions than those of the progress in Great Britain.

[2] In M. N. Baker's interesting "British Sewage Works," there is the following quotation from a pamphlet by Dr. Travis: "The conception of the Hydrolytic Tank and Oxidizing Beds is the result of a close study of the numerous experiments conducted at Lawrence under the ægis of the State Board of Health of Massachusetts, and published by that Board in a series of works which, in their entirety, constitute a classical record of the bacterial purification of sewage. This being so, an acknowledgement of the source whence the ideas were derived and a recital of the conclusions having special reference thereto are, as a matter of common honesty as well as of courtesy, equally desirable."

what he called "hydrolyzing chambers" which contained a large number of glass plates, placed transverse to the direction of flow and slightly inclined. The experiments with this apparatus convinced him that by settling out the heavy solids in one basin and then collecting a large part of the remaining undissolved organic matter on numerous vertical surfaces in another basin, the sludge thus gathered could be removed at a cost far below the expense of cleaning contact beds clogged with such material.[1]

Travis Tanks.—The information obtained by these experiments was used in designing what is commonly termed a Travis tank, although Travis called it a hydrolyzing tank. The sewage was first screened through a rack with ½-in. meshes and then passed through 2 grit chambers, each with a capacity of about 3600 gal., equal to the flow for 15 minutes at the average rate during 24 hours. The sewage was then discharged through submerged openings into the hydrolyzing tank, about 75 ft. long and 17½ ft. wide, with a semicircular bottom. From each side of this bottom a longitudinal partition sloped up and inward at an angle of 45 deg., until they were close together, and then extended vertically upward, thus dividing the tank into 3 longitudinal chambers. The side chambers were called sedimentation chambers and that in the center a liquefying chamber. At the bottom of each sedimentation chamber were narrow openings into the liquefying chamber which afforded the only entrance to the latter.

The sewage flowed from the chambers over weirs, that at the outlet of the liquefying chamber being 2 ft. long and those at the outlets of the sedimentation chambers 7 ft. each. By this arrangement of parts, 87.5 per cent. of the sewage flowed through the sedimentation chambers in 5 hours and 12.5 per cent. descended through the openings into the liquefying chamber and was in the tank for 15 hours. An essential feature of Travis' theory was that it was necessary to draw about one-sixth to one-eighth of the sewage through the liquefying cham-

[1] Dr. Travis once stated the theory upon which he worked as follows: "The Hampton doctrine supports the Dunbar absorption theory in holding that the impurities are removed from the sewage as a preliminary physical effect, but differs from that theory in holding that the property of retention plays an equally important part in sewage purification processes, and is also at variance with it in teaching the necessity for differentiating between the operations upon the solids in actual solution and those upon the solids in colloidal solution. The soluble solids are mainly removed by a process of absorption, the tendency of which is toward saturation or to the establishment of an equilibrium, an effect which would be speedily brought about in artificial treatment areas unless counteracting forces were at work removing or destroying the absorbed matter. This action, therefore, is strictly dependent upon the renovation of the absorbing surface for its effective continuance. Whereas the pseudo-dissolved solids are deposited in the filter as solid matter which, instead of demanding immediate destruction in order to insure the deposition of the putrescible matters in the next charge of sewage, remains in the filter, increases, indeed, largely forms the absorbing area, and in this way tends to the more efficient action thereof. Moreover, this depositing operation will continue uninterruptedly and, ceteris paribus, more and more completely until the filter is choked."

ber in order to seed the liquid there with fresh bacteria. The purpose of the novel arrangement of parts was to intercept the floating matter by baffles at the end of the sedimentation chambers and to wash the settling matter through the narrow openings before mentioned into the liquefying chamber. In this way the liquefaction of the sludge was expected to take place apart from the main portion of the sewage, and the gases given off during this decomposition would not, it was expected, pass through the openings into the relatively fresh sewage in the side chambers.

The effluent from the hydrolytic tank was delivered to 4 hydrolyzing chambers, arranged in sequence. Each had a bottom consisting of 3 parallel brick arches into which the effluent was delivered. The liquid rose through openings in these arches into a bed of broken flints from 3 to 6 in. in size and about 7½ ft. deep. The sediment collecting below the arches could be removed through drains. The sewage occupied about 3 hours in passing successively through the chambers and was then taken to the contact beds. The total amount of sludge collected from the hydrolyzing tank and hydrolyzing chambers was about 1900 lb. per 1,000,000 gal. during the early period of operation.

The works at Hampton attracted much attention, but only one large installation along the same lines was made after the completion of the original plant. This was carried out at Norwich, England, from the plans of City Engineer Arthur E. Collins, prepared in co-operation with Travis. A new feature was introduced by hanging in the middle three-fourths of the length of each sedimentation chamber 1.5 × 0.75-in. vertical splines of hard wood, 3 in. apart transversely and 5 to 9 in. longitudinally. Similar splines and also some wooden shutters were hung in the hydrolyzing chamber. "The function of these splines is to attract the fine non-depositable suspended solids, and to insure the coagulation of some proportion of the matters in colloidal solution. They have, therefore, been called colloid collectors, or colloiders." (Collins.)

The interest shown in these tanks was due mainly to a rather general conviction that a large part of the foulness of stale sewage was due to the putrefaction of the sludge in contact with it. Any attempt to liquefy the sludge apart from the sewage was looked upon, therefore, as a step in the right direction.[1] The septic tank had been found capable of reducing

[1] "The observation that the stronger the sewage entering a septic tank the greater the percentage removal of organic matter, suggests the idea that, where exceedingly large volumes of sewage are to be purified, . . . the sewage could be passed through ordinary settling tanks, so constructed that the sludge settling to the bottom of the tanks could be flushed into a septic tank and this sludge alone be treated by septic tank action, instead of attempting to treat the whole volume of sewage." (Rept., Mass. Bd. Health, 1899, page 422). Numerous experiments along this and related lines are recorded in later reports. Separate sludge digestion in open tanks has been carried on successfully at Baltimore, Md., and Birmingham, England.

the volume of sludge, but the latter was usually offensive and the effluent from the tank was often very mal-odorous, which was a drawback to its distribution over the top of filters, particularly by nozzles. The ebullition of gases in these tanks also resulted in a large amount of suspended matter in their effluents.

Imhoff Tank.—The operation of the Travis tank at Hampton made such an impression on Hering that when Wattenberg, the sewerage engineer of the Emscher Drainage District Board,[1] asked him during a call at his office what treatment works would furnish the most useful information to an engineering visitor, the German was advised to make a thorough study of the works at Hampton. As a result of such a study, Wattenberg began the construction of a Travis tank in 1905. His death occurred in that year, and the tank was not completed along the lines of the Travis patent. The reason for this was that, acting on the advice of Wattenberg's successor, Imhoff, the Board had adopted the policy of keeping sewage as fresh as possible from the house drain to the outlet in the river. The passage of any sewage through the reduction chamber, as is practised in the Travis tank, was considered undesirable because this sewage would become needlessly stale or septic. Accordingly the Emscher or Imhoff tank was designed to collect the settling solids in the sewage and prevent absolutely the gases and scum given off during their digestion from reaching the sewage. No attempt was made to collect the colloidal matter in the sewage, to which Dr. Travis had devoted so much attention. The solids dropped through a slot in the bottom of the sedimentation chamber into a sludge chamber below it. This slot was constructed in such a fashion that no gases could escape through it into the chamber through which the sewage was flowing.

The sludge was allowed to stay in the sludge chamber for a period ranging from a few weeks to several months, depending upon various local conditions. When in a proper condition for disposal, it could be readily removed from the tank to a sludge bed, where exposure to the air for a few days left it in a condition resembling humus. It was inoffensive in every way and the dumps where it was deposited were not nuisances in any sense. The Imhoff tank greatly reduced the cost of disposing of sludge as compared with anything accomplished under like

[1] The Emscher River flows through a district devoted to the coal and iron industry. The drainage area is about 300 square miles. The growth of the Krupp works at Essen is typical of what has taken place in other parts of the district on a smaller scale. The river became heavily polluted with sewage and industrial wastes, and finally, in 1889, the state appointed a commission to improve the conditions. It spent about $1,500,000 in draining marshes but did little to prevent pollution. In 1904 the Emscher Drainage District Board was organized to carry on sanitary and other works, for the conditions had become such, to quote Dunbar, that "the very existence of valuable industries had become intimately bound up with the question of river regulation and purification." Its sewerage work includes a large number of lined open channels and other unusual features.

conditions prior to its introduction. This fact was quickly recognized by Hering as of special importance to many American cities, where the wide annual range of temperature and heavy rainfall make sludge treatment on beds a somewhat uncertain and always difficult and expensive process. Largely through his articles and addresses, the Imhoff tank was soon in favor in the United States, and by the close of 1914, about 75 cities and many institutions had received licenses to use them.

Aeration.—At the time when the changes in sewage were regarded as chemical phenomena of the nature of oxidation, aeration of sewage to increase the rate of oxidation was tried by a number of investigators. As a rule, these experiments were made by forcing air into filters through pipes, as done by Lowcock at Malvern and Wolverhampton in 1893 and by Waring at Newport, R. I., about a year later. Col. Ducat devised a form of filter with passages through the walls, in order to afford access for air to reach the filtering medium. All this work was done with sewage in motion.

The earliest aeration work with sewage in tanks was apparently done by Lowcock.[1] Although this was promising, it was dropped for a time. Col. W. M. Black and Earle B. Phelps revived interest in such aeration by experiments at New York in 1910–1911, showing a decided improvement in the stability of sewage by it. More elaborate experiments by the Massachusetts State Board of Health and by English investigators working with the advice of Dr. Gilbert J. Fowler, of Manchester, indicate that prolonged aeration of sewage is necessary only while the tank is becoming properly ripened, and that when a sludge of a certain character has been formed in the tank, aeration for much shorter periods results in remarkably good clarification. The results obtained by the Massachusetts State Board of Health are considered very important by that body, and in its 1913 report aeration was stated to afford the best tank effluent for further treatment which had been obtained by all the methods tested at the Lawrence Experiment Station. Investigations have also been commenced by the Illinois State Water Survey, the Milwaukee Sewerage Commission, the Baltimore Sewerage Commission, the U. S. Public Health & Marine Hospital Service and other bodies. They are not far enough along at this time (April, 1915) to furnish conclusive information of a practical nature, but the early work promises important final results.

Royal Commission on Sewage Disposal.—The second Royal Commission on Sewage Disposal was appointed in 1898 as a result of strong

[1] "He had carried out some experiments a good many years ago on aerating sewage, and he had found that a very marked result was produced in precipitation. First of all he had precipitated the sewage and then passed the clarified effluent into a second tank and aerated it, then he had decanted the water from the top of the second tank into a third tank. The precipitation in the third tank had been far greater than in the second." S. R. Lowcock, Proc. Inst. C. E., vol. clxiv, part ii, 1906.

opposition to the policies of the Local Government Board regarding the necessary treatment of sewage already mentioned on page 6. It was claimed that the Board failed to realize the extent of the progress made since its investigations in 1875, and was causing cities needless expense by its insistence on final land treatment of the effluent from other processes of purification. The Commission was given three subjects upon which to report, which were worded so as to enable it to make a comprehensive study of the state of the art. The engineering profession was represented among its members, it had a staff of well-known technical specialists, it heard testimony from many persons competent to speak authoritatively on the different aspects of sewage treatment and disposal, and it carried on a number of investigations on its own responsibility. Its first report was published in 1901 and the ninth and last appeared in 1915. The detailed information obtained at the inquiries and by its specialists is of great value to the investigator. The main conclusions of the Commission appeared in 1908, in its fifth report, and were summarized in the following paragraphs:

"We are satisfied that it is practicable to purify the sewage of towns to any degree required, either by land treatment or by artificial filters, and that there is no essential difference between the two processes, for in each case the purification, so far as it is not mechanical, is chieflyeffected by means of micro-organisms. The two main questions, therefore, to be considered in the case of a town proposing to adopt a system of sewage purification are, first, what degree of purification is required in the circumstances of that town and of the river or stream into which its liquid refuse is to be discharged; and, second, how the degree of purification required can, in the particular case, be most economically obtained.

"The choice of a scheme must depend on a number of considerations which will be discussed later, but we may here state that we know of no case where the admixture of trade refuse with the sewage makes it impracticable to purify the sewage either upon land or by means of artificial processes, although in certain extreme cases special processes of preliminary treatment may be necessary"[1] (page 9).

"We find that it is generally desirable to remove from the sewage by a preliminary process, a considerable proportion of the grit and suspended matters, before attempting to purify the sewage on land or filters" (page 229).

"*Quiescent Sedimentation.*—Two or three hours quiescence is usually sufficient to produce a tank liquor fairly free from suspended solids, but owing to the fact that some sewages contain a larger proportion than others

[1] An investigation of the effect of trade wastes on sewers and sewage treatment works was made in 1914 by George S. Webster, Chief Eng. of the Philadelphia Bureau of Surveys. The results indicate, according to W. L. Stevenson, that wastes should not be admitted to sewers if they contain substances which will adhere to the walls or form deposits, if they contain inflammable substances, if they contain steam or very hot liquids, if they contain acids which will injure the material of the sewer, or if they contain substances which will seriously interfere with the operation of sewage treatment works.

of solids that settle very slowly, no general rule can be laid down as to the necessary period of quiescence. With this form of treatment the deposit in the tanks should be frequently removed.

"*Continuous Flow Sedimentation.*—The amount of settlement effected does not depend alone upon the period of flow, but upon a number of other factors. If the tank liquor is to be treated upon filters of fine material, the period of flow should generally be from 10 to 15 hours. The tanks should be cleaned out at least once a week" (page 229).

"*Septic Tanks.*—All the organic solids present in sewage are not digested by septic tanks, the actual amount of digestion varying with the character of the sewage, the size of the tanks relative to the volume treated, and the frequency of cleansing. With a domestic sewage and tanks worked at a 24-hour rate, the digestion is about 25 per cent. The liquor issuing from septic tanks is bacteriologically almost as impure as the sewage entering the tanks. Domestic sewage which has been passed through a septic tank is not more easily oxidized in its passage through filters than domestic sewage which has been subjected to chemical precipitation or simple sedimentation.

"No definite rules can be laid down as to how long a septic tank should be run without cleaning. In the case of small sewage works (serving populations of say 100 to 10,000 persons) the tanks should generally be allowed to run, without cleaning, so long as the suspended matter in the tank liquor shows no signs of affecting the filters injuriously. For larger works it would generally be advisable to run off small quantities of sludge at short intervals of time.

"The rate of flow through a septic tank is a matter in which the needs of each place require special consideration, but at few places should the sewage be allowed to take longer than 24 hours or less than 12 hours to flow through the tank. In no case should less than two tanks be provided, and they should be arranged so that, if necessary, one tank can be used alone.

"By passing septic tank liquor through tanks of a size sufficient to hold about one-quarter of the day's flow, with the addition of from 2 to 3 grains of lime per gal. (235 to 355 lb. per 1,000,000 U. S. gal.) to the liquor, the suspended solids in the liquor are materially reduced, a considerably larger quantity of the liquor can be treated per cubic yard of filter, and the offensive character of the liquor is largely destroyed" (page 229).

"*Chemical Precipitation.*—In the case of sewages which contain certain trade wastes, and strong sewages from water-closet towns, it is generally desirable to subject the sewage to some form of chemical treatment before attempting to oxidize the organic matter contained in it. In most cases careful chemical precipitation materially aids the deposition of the suspended solids and facilitates subsequent filtration. No general rule can be stated with regard to the capacity of precipitation tanks. With continuous flow, an 8-hour rate is usually sufficient to produce a fairly good tank liquor from a domestic sewage of average strength. If sewage is allowed to remain quiescent in the tank, 2 hours' settlement would usually suffice" (page 230).

"*Relative Cost of Different Tank Treatments.*—In the absence of special circumstances favoring a particular plan, it would appear that there is very

little difference in annual cost between the various methods of tank treatment when taken in conjunction with the cost of subsequent filtration through percolating filters, assuming that the kind of filter adopted in each case is that which is best adapted to the particular tank treatment provided" (page 230).

"*Contact Beds.*—Within ordinary limits, the depth of a contact bed makes practically no difference to its efficiency per cubic yard. We think that it would be generally inadvisable to construct contact beds of a greater depth than 6 ft. or of a less depth than 2 ft. 6 in." (page 230).

Trickling Filters.—"For practical purposes and assuming good distribution, the same purification will be obtained from a given quantity of coarse material, whether it is arranged in the form of a deep or of a shallow percolating filter, if the volume of sewage liquor treated per cubic yard be the same in each case.

"With regard to percolating filters of fine material, if the liquid to be purified were absolutely free from suspended and colloidal solids, and if thorough aeration could be maintained, the statement just made for filters of coarse material might possibly hold good for filters of fine material also. In practice, however, these conditions can scarcely be maintained with large rates of flow, and we think that the greatest efficiency can be got out of a given quantity of fine material by arranging it in the form of a shallow filter rather than of a deep filter. But we are not in a position to make an exact quantitative statement as to the difference in efficiency of the two forms.

"With percolating filters there is apt to be nuisance from flies, especially with filters constructed of coarse filtering material. In the warmer months of the year, such filters swarm with members of the Psychodidæ (small midges), which, though appearing to breed and develop in the filters, may usually be seen in large numbers on the walls of houses or buildings close to or on the works" (page 230).

Comparison of Contact Beds and Trickling Filters.—"The amount of sewage which can be purified per cubic yard of contact bed or of percolating filter varies, within practical limits, nearly inversely as the strength of the liquor treated. This statement is based on the assumption that the size of the material of which the filter is composed is, in each case, suitable to the character of the liquor treated, and that the material is arranged at the proper depth to secure maximum efficiency. Taking into account the gradual loss of capacity of contact beds, a cubic yard of material arranged in the form of a percolating filter will generally treat about twice as much tank liquor as a cubic yard of material in a contact bed. In the case of sewage containing substances which have an inhibitory effect upon the activity of micro-organisms, the working power per cubic yard of filter of either type may be more nearly equal. This point, however, is not clearly established.

"Percolating filters are better adapted to variations of flow than contact beds. Effluents from percolating filters are usually much better aerated than effluents from contact beds, and, apart from suspended solids, are of a more uniform character. On emptying a contact bed, the first flush is usually much more impure than the average effluent from the bed. The

risk of nuisance from smell is greater with percolating filters than with contact beds" (page 231).

"*Treatment of Sewage on Land.*—There is no essential distinction between effluents from land and effluents from artificially constructed filters. Effluents from those soils which are particularly well adapted for the purification of sewage contain only a very small quantity of unoxidized organic matter and are usually of a higher class than effluents from artificial filters as at present constructed and used. Effluents from soils which are not well adapted for the purification of sewage may often be very impure" (page 231).

In addition to carrying on the investigations which resulted in these general conclusions, the Commission attempted to devise some administrative system for bringing about the economical and efficient discharge of the duties of those responsible for the prevention of river pollution. The great variety of local conditions to be met led the Commission to recommend that preliminary jurisdiction be entrusted to local river or drainage district boards, from which an appeal could be made to a central authority, acting as a court of final jurisdiction in matters of river pollution and possessing various powers of investigation and scientific research. This proposal did not meet with the same degree of approval that was given to the technical features of the report.

Dilution.—In 1887, Hering began an investigation of methods of disposing of the sewage of Chicago which resulted in his planning the famous drainage canal that carries the city's sewage, diluted with a large volume of water from Lake Michigan, into the Illinois River. This recommendation was based mainly on European views regarding the capacity of flowing water to receive sewage without a nuisance being created. The sewage and industrial wastes of the city had turned the Chicago River into an open sewer of a very offensive character at times, and the proposal to reverse the direction of its flow and send all this foul matter southward through the state aroused considerable opposition. This canal is the greatest example of works for disposal by dilution, and it is questionable if it would have been practicable but for the fact that the entire undertaking lay within the Illinois boundary line. Even at that, the city of St. Louis, backed by the state of Missouri, attempted to close the canal as a menace to the health of its people, and while it lost in this litigation, the decision left the city in a position to bring suit again. The expert testimony given in this action was a thorough review of the opinions at that time of disposal by dilution, and was so important that the U. S. Geological Survey printed an abstract of it in "Water Supply Paper No. 194."

About the time the condition of the Chicago River became serious, several Massachusetts streams showed signs of serious pollution. The

State Board of Health was accordingly directed to make an examination of all the waters of the state, to ascertain if they were suitable for sources of domestic water supplies. This work was done under the direction of Dr. T. M. Drown, Chemist of the Board. In a special report on this investigation, published in 1890, F. P. Stearns, Chief Engineer of the Board, discussed the nature of the pollution of streams and of their so-called self-purification. In 1897 the Ohio State Board of Health made an investigation of the condition of the streams under its jurisdiction, and the information thus obtained was analyzed by Allen Hazen in a report bearing on the permissible contamination of rivers. In 1902 a second investigation was made by the Massachusetts State Board of Health, and its Chief Engineer, X. H. Goodnough, made an estimate of the dilution shown by this examination to be necessary to prevent offensive conditions. In 1908 a special examination of the Merrimack River was made by the State Board of Health. These investigations are the basis of current (1915) American opinions regarding the dilution necessary to prevent nuisance.

The effect of dilution upon sewage has been studied carefully in a number of American cities where expensive methods of treatment have been proposed. The case of Baltimore was peculiar, because it was felt that the existence of the important shellfish industry of the city and neighboring districts warranted the adoption of every method of treatment necessary to insure beyond reasonable question the freedom of the Chesapeake Bay oysters from infection by sewage. There are few places where the industry is of sufficient importance to be considered in determining the best method of treatment, for the removal of the industry to other localities will usually entail less expense to the public than the construction and operation of works for treating the sewage so as to avoid any real danger of infection of the shellfish. In New York very marked differences of opinion exist regarding the extent to which it is proper to rely upon the dilution of screened and settled sewage as a means of disposal. In Boston dilution is considered by the State Board of Health all that is necessary. In Providence chemical precipitation was not considered an adequate protection of the shellfish of Narragansett Bay, and in consequence disinfection of the effluent with bleaching powder has been introduced. At Rochester, N. Y., the State Health Commissioner authorized the discharge of sewage into Lake Erie after screening and sedimentation. In order that the sewage shall not cause an offensive appearance of the surface of the water receiving it, it has been proposed to discharge it through a number of small orifices rather than one large one; this method of dispersion has not yet (1915) been carried out on any large sewerage work. The only treatment needed before discharge in many cases for some years will be for clarification,

occasionally supplemented by disinfection.[1] In the case of small cities on large bodies of water, the necessity for any other treatment than dilution exists at present in very few cases. These statements indicate the authors' high opinion of the importance of well-planned dilution.

In Great Britain and Europe the subject of dilution was viewed for many years in a rather narrow way. In 1864 the Vienna Water Supply Commission fixed certain standards for the quality of drinking water, and the interest which sanitarians took in the work of the Commission was reflected by a tendency to establish standards of chemical quality for all waters. Sir Edward Frankland was influential in bringing about in England the recommendation by the Rivers Pollution Commission of 1868 of minimum standards of chemical quality for trade wastes discharged into rivers. His influential position in the scientific world led to other uses of standards of this character and the practice still persists, the latest conspicuous instance being the recommendation in 1912 by the Royal Commission on Sewage Disposal of minimum standards for the effluents of sewage treatment works.

The Germans early adopted various standards, but abandoned them as the researches of von Pettenkofer and others showed that the use of water for diluting sewage and its use as a source of drinking water presented wholly different problems, not properly subject to a single standard. The former is essentially a local problem in every case, while the latter is one in which a general standard of quality may properly be used. In the former the object of the engineer is to require the expenditure of only enough money to bring sewage into a condition which will enable the remaining steps in its cycle of change to take place in the river without causing offense. In other words, he saves money for the city by using the stream as a part of the facilities for treating the sewage. The best use of the stream will depend upon its size and character and the quantity of sewage to be treated, which vary with every problem. Water for potable purposes is always used in the same way, however, and standards of quality are as useful in one place as another in judging the fitness of a stream to be used as a source of water supply. The subject of standards will be mentioned later.

Disinfection.—With the development of knowledge of the transmission of diseases by sewage and water, the disinfection of sewage received an increasing amount of attention. Diseases of the intestinal tract, such as cholera, typhoid fever and dysentery, were discovered to be particu-

[1] "In the present state of knowledge, and especially of bacteriology, it is difficult to estimate these dangers with any accuracy, and it seems quite possible that they should be either exaggerated or undervalued according to the predisposition of those who have to deal with them. An authority, guided by medical considerations, might not unnaturally be inclined to insist on a degree of purity which may ultimately prove in certain cases to be uncalled for, while another authority, with its mind fixed upon economy, might shrink from taking essential precautions." Royal Commission on Sewage Disposal (1901).

larly liable to occur in epidemics which were traceable to the pollution of water supplies by sewage containing the germs of these diseases. While the careful filtration of infected water was found to render it safe in some instances, as at Altona, Germany, during the cholera epidemic of 1892, which proved so fatal at the neighboring city of Hamburg, some sanitary authorities claimed that there were conditions which made it desirable to kill all the germs of diseases in sewage before it was discharged into some waters. This was true not only of waters from which domestic supplies were drawn, but also those of bathing places and those utilized in the shellfish industry.

For such disinfection it was natural to turn to bleaching powder, chloride of lime, which had been long used as a disinfectant and many years ago to reduce the odor of sewage, and also to sulphate of copper, which had proved destructive of algæ and other organisms causing offensive odors or tastes in water supplies. The contamination of oysters by sewage was particularly feared by the health officers of New York, New Jersey and New England, and special attention was given to the treatment of sewage turned into tidal waters containing shellfish layings. The problem was one of bacterial rather than chemical improvement, and the desired results were attained in many cases by sedimentation and the addition of a small quantity of the sterilizing agent. In other cases more elaborate treatment was adopted, but usually the disinfection was used as a supplement to the treatment needed to prepare the sewage for its dilution in an inoffensive manner by the body of water receiving it,[1] rather than as a substitute for any form of sedimentation or filtration. The distinction is an important one, because there are some grounds for believing that carefully managed disinfection without expensive treatment of other kinds may prove in some cases to be an economical means of preventing the serious contamination of water supplies.

As experience has been gained with sewage disinfection, it has unfortunately become evident that careful management of the apparatus is not likely to be maintained in small towns. Unless it is reasonably certain that the disinfection will be properly conducted, it is questionable whether the engineer is justified in placing much reliance upon it, however economical it may be theoretically. The automatic apparatus for applying the solution of the disinfectant has proved liable to become

[1] "I have had experiments carried out with a cholera-like vibrio, which, as regards resistance, is quite comparable to the extremely sensitive cholera vibrio. These micro-organisms are specially suitable for such experiments, because they phosphoresce, and hence the colonies may be easily recognised on plate cultures. They can, however, only be used as an indicator in cases where it is a question of destroying cholera vibrios, or other equally sensitive micro-organisms; they are not so resistant as typhoid bacilli. Our experiments showed that these micro-organisms remained active for 33 days in a septic tank and only ceased to be recognisable after this period." Dunbar's "Sewage Treatment," page 232.

clogged, and must be inspected frequently. While practical sterilization is possible at a not prohibitive cost, it should not be forgotten that if sewage escapes undisinfected through storm overflows, or if the storm run-off from streets, often very impure from a bacterial viewpoint, is discharged into the body of water which it is desired to keep uncontaminated, the object of disinfection is defeated. The Royal Commission on Sewage Disposal has called special attention to the danger of a false sense of security being spread by disinfecting effluents.

Sewage Treatment and Water Purification.—It was inevitable for a great difference of opinion to arise concerning the treatment of sewage prior to its discharge into bodies of water from which municipal supplies were drawn. Health officers naturally insisted that the sewage should be rendered substantially innocuous before its discharge into such waters, on the ground that any less thorough treatment would endanger the health of consumers of the water. Many engineers believed that such requirements were too severe in most cases, because thorough treatment was required merely to reduce bacteria, this treatment was expensive and took no advantage of the beneficial effect of dilution on the sewage, the water was liable to contamination by the rain water running off cultivated lands and by the sewage from hamlets and isolated farms, and the water could be rendered safe for drinking purposes by relatively inexpensive purification. The general answer made to this was that the continuous efficient operation of water purification works was rarely attained except in large plants, and that the proper procedure was to require both sewage treatment and water purification.

The tendency in all these discussions has been to base broad general arguments on rather limited data. The great differences in local conditions render such arguments of little value as a guide for the policy any city should adopt. At Columbus, Ohio, for instance, the object of the construction of sewage treatment works was to prevent offensive river conditions within the city limits. The destruction of disease germs was unimportant. The sewage treatment works of Worcester, Mass., were built to prevent offensive conditions in the Blackstone River below the city, which was so polluted by industrial wastes and sewage from other communities that it was everywhere unsuitable for domestic water supply. At Fitchburg, Mass., elaborate treatment works have been constructed for reasons similar to those that governed the Columbus case. Such cases are different from those arising when sewage is discharged into a stream or lake from which drinking water is obtained within such a distance from the sewer outlet that its contamination is probable without careful treatment of the sewage to reduce the number of bacteria of disease.

The subject was submitted by the International Joint Commission on the Pollution of the Boundary Waters to a number of engineers, and

as a result the following statement was prepared in 1914 by George W. Fuller, George C. Whipple, Earle B. Phelps, W. S. Lea and T. J. Lafreniere:

"1. Speaking generally, water supplies taken from streams and lakes which receive the drainage of agricultural and grazing lands, rural communities and unsewered towns are unsafe for use without purification, but are safe for use if purified.

"2. Water supplies taken from streams and lakes into which the sewage of cities and towns is directly discharged are safe for use after purification, provided the load upon the purifying mechanism is not too great and that a sufficient factor of safety is maintained, and, further, provided the plant is properly operated.

"3. As, in general, the boundary waters in their natural state are relatively clear and contain but little organic matter, the best index of pollution now available for the purpose of ascertaining whether a water purification plant is overloaded is the number of B. coli per 100 c.c. of water, expressed as an annual average and determined from a considerable number of confirmatory tests regularly made throughout the year.

"4. While present information does not permit a definite limit of safe loading of a water purification plant to be established, it is our judgment that this limit is exceeded if the annual average number of B. coli in the water delivered to the plant is higher than about 500 per 100 c.c., or if in 0.1 c.c. of the water B. coli is found 50 per cent. of the time. With such a limit the number of B. coli would be less than the figure given during a part of the year and would be exceeded during some periods.

"5. In waterways where some pollution is inevitable and where the ratio of the volume of water to the volume of sewage is so large that no local nuisance can result, it is our judgment that the method of sewage disposal by dilution represents a natural resource and that the utilization of this resource is justifiable for economic reasons, provided that an unreasonable burden or responsibility is not placed upon any water purification plant and that no menace to the public health is occasioned thereby.

"6. While realizing that in certain cases the discharge of crude sewage into the boundary waters may be without danger, it is our judgment that effective sanitary administration requires the adoption of the general policy that no untreated sewage from cities or towns shall be discharged into the boundary waters.

"7. The nature of the sewage treatment required should vary according to the local conditions, each community being permitted to take advantage of its situation with respect to local conditions and its remoteness from other communities with the intent that the cost of sewage treatment may be kept reasonably low.

"8. In general, the simplest allowable method of sewage treatment, such as would be suitable for small communities remote from other communities, should be the removal of the larger suspended solids by screening through a ¼-in. mesh or by sedimentation.

"9. In general, no more elaborate method of sewage treatment should be

required than the removal of the suspended solids by fine screening or by sedimentation, or both, followed by chemical disinfection or sterilization of the clarified sewage. Except in the case of some of the smaller streams on the boundary, it is our judgment that such oxidizing processes as intermittent sand filtration and treatment by sprinkling filters, contact beds and the like are unnecessary, inasmuch as ample dilution in the lakes and large streams will provide sufficient oxygen for the ultimate destruction of the organic matter.

"10. Disinfection or sterilization of the sewage of a community should be required wherever there is danger of the boundary waters being so polluted that the load on any water purification plant becomes greater than the limit above mentioned.

"11. It is our opinion that, in general, protection of public water supplies is more economically secured by water purification at the intake than by sewage purification at the sewer outlet, but that under some conditions both water purification and sewage treatment may be necessary."

There is an apparent discrepancy between the statement in paragraph 2, concerning the practicability of purifying contaminated water, and the requirement in paragraph 6, that no sewage should be discharged without treatment into the boundary waters. This means, in effect, that a town enjoying, because of fortunate position and local conditions, a natural opportunity to dispose of its sewage with minimum expense, and without injury to other towns, must nevertheless adopt some method of treatment, in order that "sanitary administration" may be effective. While it is an old rule in law that the public welfare of many persons warrants an injury to a few, it is open to question whether this doctrine would be held by the courts to apply in cases where the gainer was merely an administrative bureau.

A standard of purity of sewage works effluents was recommended in 1912 by the Royal Commission on Sewage Disposal, which devoted its eighth report to the subject. Its conclusions were stated as follows:

"The law should be altered so that a person discharging sewage matter into a stream shall not be deemed to have committed an offense under the Rivers Pollution Prevention Act, 1876, if the sewage matter is discharged in a form which satisfies the requirements of the prescribed standard.

"The standard should be either the general standard or a special standard which will be higher or lower than the general standard as local circumstances require or permit.

"An effluent in order to comply with the general standard must not contain as discharged more than 3 parts per 100,000 of suspended matter, and with its suspended matters included must not take up at 65°F. (18.3°C.) more than 2 parts per 100,000 of dissolved oxygen (in 5 days). This general standard should be prescribed either by statute or by order of the Central Authority, and should be subject to modifications by that Authority after an interval of not less than 10 years.

"In fixing any special standard the dilution afforded by the stream is the chief factor to be considered. If the dilution is very low it may be necessary for the Central Authority, either on their own initiative or on application by the Rivers Board, to prescribe a specially stringent standard, which should also remain in force for a period of not less than 10 years.

"If the dilution is very great the standard may, with the approval of the Central Authority, be relaxed or suspended altogether. Our experience leads us to think that, as a general rule, if the dilution, while not falling below 150 volumes, does not exceed 300, the dissolved oxygen absorption test may be omitted, and the standard for suspended solids fixed at 6 parts per 100,000. To comply with this test no treatment beyond chemical precipitation would ordinarily be needed. If the dilution, while not falling below 300 volumes, does not exceed 500, the standard for suspended solids may be further relaxed to 15 parts per 100,000. For this purpose tank treatment without chemicals would generally suffice if the tanks were properly worked and regularly cleansed. These relaxed standards should be subject to revision at periods to be fixed by the Central Authority, and the periods should be shorter than those prescribed for the general or for the more stringent standards.

"With a dilution of over 500 volumes all tests may be dispensed with and crude sewage discharged, subject to such conditions as to the provision of screens or detritus tanks as might appear necessary to the Central Authority."

It will be observed that this standard pays no attention to the bacterial nature of the effluent, and it is expressly exempted from application to storm water. The Commission's recommendation regarding the treatment of storm water was given in Volume I, page 34, footnote. Its views regarding the bacterial nature of the effluent were stated as follows in its fifth report:

"We are satisfied that rivers generally, those traversing agricultural as well as those draining manufacturing or urban areas, are necessarily exposed to other pollutions besides sewage, and it appears to us, therefore, that any authority taking water from such rivers for the purpose of water supply must be held to be aware of the risks to which the water is exposed, and that it should be regarded as part of the duty of that authority, systematically and thoroughly, to purify the water before distributing it to their customers. Apart from the question of drinking waters, we find no evidence to show that the mere presence of organisms of a noxious character in a river constitutes a danger to public health or destroys the amenities of the river. Generally speaking, therefore, we do not consider that in the present state of knowledge, we should be justified in recommending that it should be the duty of a local authority to treat its sewage so that it should be bacteriologically pure."

A number of the leading English authorities on sewage treatment, including Watson of Birmingham, Fowler of Manchester, Hart of Leeds,

Garfield of Bradford and Mawbey of Leicester, prepared in 1913, for the Association of Municipal Corporations, an adverse discussion of the proposed standards. It was held by them that any new legislation should be designed to conserve or improve the condition of the streams and not to compel cities to produce sewage effluents of uniform quality. It was pointed out that no two cases are precisely similar, and it was claimed that instead of establishing standards it would be better to give to some suitable board, possessing technical knowledge and power, authority to hear and settle all disputes regarding the quality of sewage effluents and the pollution of water. Up to 1915, Parliament had not acted on the subject.

In Germany, where bureau control is developed very thoroughly, a long experience with various arbitrary regulations concerning the treatment and disposal of sewage has resulted in a recognition of the essentially local character of such work. Dr. Dunbar summarizes this opinion, in his "Principles of Sewage Treatment," as follows:

> "The question of sewage disposal is intimately bound up with that of spreading epidemics by means of polluted rivers. The danger of infection may, however, be dealt with separately from the problem of sewage purification, and such a treatment of the subject is desirable, because the problem is often only complicated by the introduction of questions of infection in cases where the possibility of infection is very remote. Each case will have to be dealt with individually according to whether or not those living lower down the stream are dependent upon the water of the stream for drinking and other domestic purposes, and the measures to be adopted in cases of epidemic disease can only be decided upon after due consideration of all the local conditions" (page 41).

Sewage Disposal in Germany.—There were comparatively few rivers in Germany which were seriously polluted until recently, and the early instances of marked pollution were due mainly to industrial and mining wastes. Sewers were constructed very slowly in comparison with the laying of water mains, because the cost of the latter could be covered by the water rates while the sewerage works were built by a general city tax, and the German cities were not in good financial condition in the middle of the last century.

Although sewerage construction on a comprehensive, well-considered plan began in Germany in 1842, in Hamburg, it was 18 years before the second undertaking of this nature was started. This was at Frankfort. Both cities engaged W. Lindley, an English engineer, to design the sewers and superintend their construction. About the same time sewerage systems were begun at Stettin under Hobrecht, who later planned the radial sewers of Berlin, and at Danzig under E. Wiebe. The sewerage of Breslau was begun in 1875, of Carlsruhe in 1877 and of Munich in 1880. In many cities, of which Bremen is a typical example,

the pail system was used, and in others cesspools were employed. In some places where the latter had been adopted, an extra charge was made for cleaning cesspools into which water-closets discharged, owing to the opinion that the water used to flush them reduced the fertilizing value of the contents of the cesspools. Even after sewers had been built in many German cities, cesspools remained in use and the per capita discharge of fecal matter through the sewers was very small.[1] The German rivers are larger than those of England and the discharge into them of small quantities of relatively weak sewage, compared with that of English cities, did not have the same offensive results that followed the pollution of British streams.

Another influence in fixing the character of German sewage treatment is the division of authority over rivers. The imperial authorities are not permitted by the federal constitution and civil code to exercise jurisdiction over interstate waters, except in certain features relating to health. The early local regulations were based largely on British opinions of that period. Irrigation was adopted for Breslau and Berlin, where the land was unusually favorable for treating sewage, and chemical precipitation was employed at Frankfort, Halle, Essen, Leipzig and other places. The effluents from some treatment plants failed to meet the official requirements, although every effort was made to produce the required results. During this period of discussion and earnest endeavor to comply with the law, it became apparent that these effluents were not causing any nuisance in the streams into which they were discharged. When this fact was established the strict regulations were abandoned and each disposal problem has since been considered independently. The problem of preventing nuisance has been divorced from that of disinfecting sewage. Provision must be made for disinfecting all sewage liable to be infected, whether it be from a single house or an entire city, but this disinfection is carried on only while there is danger of spreading some water-borne disease through the medium of the sewage.

To prevent offensive conditions in the rivers receiving sewage, the German practice after the relaxation of severe, impracticable requirements was at first toward clarification in settling basins. Some of the experiments to determine the best shape of such tanks, notably those conducted about 1901–1902 by Steuernagel at Cologne and Bock and Schwartz at Hannover furnished information of much value to en-

[1] In 1907, of 719 German cities with populations exceeding 5000, 193 (27 per cent.) had good sewerage systems, 63 (9 per cent.) had partial systems, 151 (21 per cent.) had storm-water drains, 138 (19 per cent.) had more or less complete systems in view, and 174 (25 per cent.) had no system built or proposed. In 1909, of 643 French cities with more than 5000 inhabitants, 320 (50 per cent.) had no sewers whatever, 257 (40 per cent.) had storm-water drains only, and 66 (10 per cent.) had complete storm-water drains and sewerage systems. Only 4 of the 66 cities had the separate system. (*Eng. News*, April 21, 1910.)

gineers. Even earlier Müller and Nahnsen introduced (1885–1886) at Halle deep circular sedimentation tanks, which were adopted by Kniebühler at Dortmund in 1887. The installation at the latter city became a model on which many designs have been based.

The construction of tanks was sometimes difficult on account of lack of space or bad foundations, and about the same time several engineers turned their attention to the possibility of securing the desired degree of clarification by passing the sewage through fine screens. These were found to furnish promising results, and a number of radically different moving, self-cleaning types were developed. Some were endless belts of wire mesh, others were racks of small bars jointed together to form endless link-belts, others were fine racks held by arms projecting from a shaft so that the arrangement was like that of a paddle-wheel, others were drums covered with wire mesh, and others were perforated disks revolving about an axis inclined so that the lower part of the disk was submerged in the sewage. The actual performance of each of these fine screens is not yet well established, but they are considered to meet the requirements as a rule and the approval they have received in Germany has led to their introduction in the United States. The experience gained here with the drum type has not been particularly encouraging, although this may have been due in part to mechanical imperfections, and other types have not yet (1915) been tried sufficiently to furnish enough information to warrant drawing any conclusions as to the degree to which they may be regarded as substitutes for or complements to settling basins.

No mention of German sewage disposal would be complete without reference to the research work done at the Hamburg State Hygienic Institute under the direction of Dunbar, and to that carried on at the Royal Testing Institute for Water and Sewage at Berlin, under the direction of Dr. A. Schmidtmann. At the former, the scientific basis of sewage treatment has received much study and the results have been utilized in numerous works built under the advice of the director. Although long a resident of Hamburg, Dunbar is an American by birth and early education; his "Principles of Sewage Treatment" is one of the leading works on the subject. The Berlin Institute was established in 1901 to collect the scientific information necessary for a proper discharge of the duties regarding sewage treatment imposed upon local authorities. These duties comprise checking the spread of disease by water-borne organisms, the prevention of water contamination where health is endangered, the prevention of offensive conditions and the protection of useful fishes. The Prussian regulations regard each case as a local problem and the Institute furnishes the information, when it is needed, for a judgment on the scientific questions involved.

Site and Cost of Treatment Works.—The selection of the best method of treatment of the sewage of a community depends not only on the requirements arising from the condition of the water into which the effluent will be discharged, but also on the practicability of obtaining sites for the works proposed by alternative plans and the cost of constructing and running these plants. It may even happen in a large city that two or more treatment plants will be better than one. R. W. Pratt recommended three plants for Cleveland in 1915, for example, although their cost and that of the necessary sewers to feed them is about the same as that of the original plan contemplating one outlet. One plant will have grit chambers, sedimentation tanks and trickling filters; another will have grit chambers, sedimentation tanks and disinfecting apparatus; the third will have grit chambers, sedimentation tanks of small capacity and disinfecting apparatus. By this plan the sewage will be in a fresh condition when treated and it is expected that both the cost of the treatment and danger of odors will be reduced.

In all such problems, dilution as a means of treatment and disposal should be considered. The joint trunk sewers serving many of the cities and towns about Boston, Mass., and Newark, N. J., are examples of works constructed co-operatively, which relieve inland communities of treatment problems that would be quite serious in some cases. At Los Angeles, after an unsuccessful experience with sewage irrigation, it was considered best to construct an outfall sewer, 12.4 miles long, to the ocean.

In compiling figures for judging the relative merits of different methods of treatment, the cost of pumping and of a trunk sewer to convey sewage to the works and an outfall sewer from the works must be considered a part of the total expense, and sometimes these items are a large part of the entire amount. Interest, maintenance and sinking-fund charges on such structures must be combined with the fixed, operating and maintenance charges on treatment works in order to reach a true estimate of the annual cost of the method of treatment.

In preparing estimates of cost, the provision of land for extensions to meet future requirements must receive attention. This is particularly important where the treatment includes intermittent filters or other structures of large area in proportion to their capacity, and where it is probable that the degree of purification effected by the works must be increased with the lapse of time. Overworking treatment plants which it is impracticable to enlarge usually results in a nuisance, and no plans for sewage treatment should be adopted which do not provide for a proper increase of capacity during the period for which the works are designed. This period may be somewhere between 25 and 40 years, depending on local conditions. For example, it may be reasonably certain that

clarification of sewage by settling and sedimentation, with disinfection of the effluent, will be a satisfactory treatment for 25 years, and enough land is available near the city for works adequate for the requirements during that period, but no longer. Such a plant may prove more economical than the construction of a trunk sewer to a site much farther away but large enough for works of several times the utmost capacity of those nearer the city. It is true that the works close at hand may be serviceable for only a comparatively short period, but their total cost during that period may be less than the total cost during the same period of the works at a more distant site. It is hardly possible to estimate closely the future requirements of a city for more than 35 or 40 years, and in such a period of time sewage treatment may progress in many ways, so that it is unwise to plan works "for all time."

Legal Aspects of Sewage Disposal.—The pollution of water has given rise to a large amount of litigation. In the case of unnavigable running streams, the law as interpreted in most states is based on the old common-law principles, (*a*) that every riparian owner is entitled to have the waters of the neighboring stream reach his property in their natural condition except for any reasonable use of them by upper riparian owners, and, (*b*) that every riparian owner is entitled to make such reasonable use of the water flowing past his property as he sees fit. In the semi-arid regions[1] the legal doctrine of prior appropriation has supplanted that of reasonable use, just stated, and litigation regarding the pollution of waters in those regions has not been sufficiently extensive and important to show how far, if at all, the law as regards river contamination in the Eastern States will be changed by the courts of final jurisdiction of the states where the common-law view of water rights does not prevail.

The two water rights of a riparian owner, under the common law, hinge on the interpretation of what is meant by a "reasonable" use of the water. For many years the courts have been engaged in settling suits involving that question, and today it is established in many states that a riparian owner can freely use the water for watering stock, household purposes and irrigation of land, provided these uses make no appreciable reduction in the volume of the stream and result in no contamination of its waters. It is evident, however, that what might be considered reasonable use of the stream for fishing, drainage of agricultural lands, removal of sawmill refuse and other purposes in sparsely settled regions would be prejudicial to public welfare in more populous districts. Hence the courts have adopted no rigid rules for interpreting the law, but have decided what was reasonable upon the local merits of each case.

[1] The doctrine of prior appropriation is in force in Arizona, Idaho, Montana, Nevada, New Mexico, North Dakota and Wyoming, and in many other states in the arid districts it is partly recognized.

The principles of the common law are also applied to the waters of privately owned, natural ponds. If there are several owners, each has the same rights as the others, just as though the pond were a stream and the owner a riparian owner. In the case of ponds larger than 10 acres, in some New England states, there is a complication due to very old statutes setting aside such ponds as public property for fishing and hunting. In Massachusetts every citizen has right of access to such ponds, except when prevented by subsequent legislation, over any private property except meadow and corn land. Although these statutes were repealed long ago, they were in force long enough for the principle to be established as common law in the states in question. The public rights to some of these "great ponds" have been given away by legislatures, but there are others in which all citizens still have equal rights. But if sewage is turned into such a pond to the injury of the citizens using the water for any legal purpose, none of them has any means of recovering damages for this infringement of his rights, such as he possesses in the cases previously mentioned, but only the state has any remedy. This is because such conditions are governed by the principle that no private citizen has any remedy for a public nuisance. The pollution can be stopped, under the common law, only by indictment, although private parties have a different remedy based on equity and not on law. (Chas. F. Choate, Jr., *Jour. Assn. Eng. Socs.*, 1908, vol. xl, page 60.)

The invasion of public and private rights in ponds and non-navigable streams by turning the contents of sewerage systems into them is not due to the collection of storm water in drains and sewers and its discharge into the natural drainage courses of the catchment area. Storm water, no matter how much increased in volume by a reduction in the extent of permeable ground and changed in character by the inclusion of street refuse, is legitimately discharged into the streams naturally carrying away the run-off of the valley.

The case is different with sewage, but it is necessary even here to draw a distinction between two possible views of the nature of damages due to sewage pollution. In the first, or legal, aspect of the conditions, the city is the agent of the state, acting under legislative authority given in a charter or special act. As such an agent of the state, it has been held in Massachusetts to be without liability in damages to a lower riparian owner because of its discharge of sewage into a stream. In the second, or equity, aspect of the conditions, the owner would not ask damages but would endeavor to prove, as has been done in Connecticut that the discharge of the sewage was a continuing injury to his property unwarranted by any statute. If this contention were upheld the court would probably grant an injunction preventing the continuance of the pollution. The distinction between the legal and equity remedies for

stream pollution is fundamental. It is quite generally regarded that where manifest pollution of a stream has not occurred for more than 20 years[1] or is not authorized by statute, it can be stopped by equity proceedings, but the outcome of suits for damages under the common law is more uncertain. It makes little difference whether the pollution of the stream is due to domestic sewage or industrial wastes, so long as it infringes upon the rights of a riparian owner to the reasonable use of the water in its natural condition. Only statutes and prescriptive rights acquired by 20 years' usage can give authority to pollute water.

The pollution of tidal waters is different legally, and has not been placed on an even moderately satisfactory basis. This is because ownership of land on tidal water does not confer any right upon the owner to have the ocean brought to his property without contamination. In some states private ownership extends to the low-water contour and in others only to the high-water contour. Land below these contours limiting private ownership and the tidal waters themselves are public property. The discharge of sewage into these tidal waters may or may not be a nuisance or menace to health. Apparently no owner of water-front property can have a private action for damage in such a case unless his land has been actually damaged, but there is little doubt that any pollution which reaches the condition of a public nuisance can be stopped.

In the case of danger to public health, the common law, which is concerned with property rights, is best supplemented by statutes as the extent to which the former will apply to infractions of sanitary principles is uncertain. The statutes may either be passed directly by a legislature, or the latter may clothe a commission with powers to make regulations which become, for all practical purposes, the same as statutes.

The legal aspect of water contamination is discussed in the "Pollution of Inland Waters," by Edwin B. Goodell (Water Supply Paper 152, U. S. Geol. Survey, 1905) where special attention is given to statutory laws on the subject. His conclusions regarding the rights and duties of municipal corporations are substantially as follows:

Considered as corporations, municipalities have only such rights and powers as are conferred on them by statute, either expressly or by necessary implication. When, under due authority, they become the owners of lakes, reservoirs and natural streams, they have the same rights to pure water, and are charged with the same duties, as other riparian proprietors. If authorized to construct systems of sewers draining into streams, such authority does not exempt them, except in Indiana, from the duty not to pollute the stream to the damage of lower proprietors. The common-law water rights of property owners cannot be taken

[1] Even prescriptive rights are not recognized in some states, as Connecticut, to sanction the discharge of sewage into streams when it constitutes a public nuisance.

from them for public use except upon payment of an amount determined by condemnation proceedings authorized by statute. Until municipal corporations have acquired by such proceedings the rights of all lower proprietors and paid for them, they are required in all cases to refrain from the pollution of streams, to the same extent as private owners.

A list of opinions by courts of final jurisdiction in cases involving sewage disposal would be too long for publication here, but as engineers are occasionally asked to furnish references to such opinions the following cases are cited:

Edmondson *v.* City of Moberly, Mo., 11 S. W. Rep. 999; Seifert *v.* City of Brooklyn, 4 N. E. Rep. 321; Morse *v.* City of Worcester, 2 N. E. Rep. 694; Railroad Co. *v.* Baptist Church, 2 Sup. Ct. Rep. 719; Joplin Consolidated Mining Co. *v.* City of Joplin, 27 S. W. Rep. 408; Locks and Canals *v.* City of Lowell, 7 Gray, Mass., 223; Haskell *v.* City of New Bedford, 108 Mass. 208; Vale Mills *v.* City of Nashua, 63 N. H. 136; Chapman *v.* City of Rochester, 18 N. E. 88; Peterson *v.* City of Santa Rosa, 51 Pac. Rep. 557; People *v.* City of San Luis Obispo, 48 Pac. 723; Good *v.* City of Altoona, 29 Atl. Rep. 741; Missouri *v.* Illinois and Sanitary District of Chicago, 21 Sup. Ct. Rep. 331; Platt Bros. & Co. *v.* City of Waterbury, 45 Atl. Rep. 154; Nolan *v.* New Britain, 38 Atl. Rep. 707; Pumpelly *v.* Green Bay Co., 80 U. S. 177; Carmichael *v.* City of Texarkana, 94 Fed. Rep. 561; Parker *v.* Am. Woolen Co., 81 N. E. Rep. 468, 215 Mass. 176. The Indiana decisions differ from those quoted and have been found difficult to reconcile with each other (Markwardt *v.* City of Guthrie, 90 Pac. Rep. 26); typical cases are City of Valparaiso *v.* Hagen, 54 N. E. Rep. 1062; City of Valparaiso *v.* Moffit, 39 N. E. Rep. 909; City of Richmond, *v.* Test, 48 N. E. Rep. 610.

CHAPTER II

MEANING OF CHEMICAL ANALYSES

The character of sewages, of effluents from sewage treatment works and of natural waters is determined by chemical, bacterial, and microscopic analyses and tests, although one of experience can learn much by mere observation of the physical qualities of the liquids.

Chemical analysis gives directly such information as the actual content in sewage of nitrogen, potash and phosphorous compounds, which have a definite monetary value in the market and from which the theoretical fertilizing value may be computed. Other information furnished is of assistance in a relative or comparative way, enabling the chemist to form certain opinions through a comparison of the results of one analysis with those of many others. An illustration is furnished by a river water, found on analysis to contain certain ingredients, in quantities much larger than are found in other similar, neighboring waters, thus proving sewage pollution. Chemical analysis also furnishes data from which an opinion may sometimes be formed as to what changes will take place in water or sewage under natural or artificial conditions. As an illustration certain analyses and tests enable the chemist to predict whether a water will change so as to produce offensive odors under the conditions to which it will probably be subjected.

Many sanitary investigations as well as the successful operation of sewage treatment plants and sometimes the intelligent control of dilution projects, require chemical analyses. For the benefit of engineers, lawyers and others who lack a knowledge of chemistry adequate to a full understanding of the results of these analyses, an effort is made in this chapter to explain the general method of analysis, the meaning of the technical terms used and the interpretation of the results obtained.

Importance of Proper Sampling.—If the interpretation of the results of an analysis is to be accurate it is of first importance that the sample analyzed be representative of the waters from which it is taken. Sewage varies greatly in quality from hour to hour, from day to day, and from season to season. It is, therefore, necessary in making a study of the character of a sewage to analyze a sufficient number of samples. To determine the general character of a sewage for an entire day it is best to take samples every half hour throughout the 24 hours and combine them in one sample to be analyzed. The quantity of sewage also varies greatly from hour to hour, as explained in Chapter V, Volume I, and it

is, therefore, necessary, if the composite sample is to be strictly representative, to make the individual samples proportionate in quantity to the volume of sewage flowing at the time of sampling.

Most sewage contains large quantities of floating or suspended matter, some of which is quite coarse. In order that the composite sample may be as nearly as possible representative of all of the different matters in the sewage it is desirable that the samples taken should be relatively large. For sampling purposes, it is often well to take 1 quart or more every half hour, pouring it into a clean tub or barrel; and to mix the contents of the barrel thoroughly at the end of the day and take from 2 to 4 quarts of the composite for analysis. Precautions have to be taken in the laboratory also to be sure that the small samples taken for tests are fairly representative of the large sample sent in.

Preservation of Samples.—Waters undergo physical, chemical and biological changes when allowed to stand. It is, therefore important that the analysis be made as soon as possible after the sample is collected. Sewage decomposes very rapidly on standing for a comparatively short period of time at room temperature. The time which may be allowed to elapse between sampling and analysis depends largely upon the degree of pollution of the sample under examination. Fairly pure ground and surface waters may safely be allowed to stand 2 or 3 days before the chemical analysis is made. Polluted waters should ordinarily be analyzed within 12 hours, and raw sewages and sewage effluents preferably within 6 hours. Certain determinations, such as that for dissolved oxygen, should be made on the spot.

The rate of decomposition of sewage and polluted waters may be lessened materially by keeping the samples at a low temperature, as in an ice box. Even at this temperature, however, changes will take place. If the samples are to be shipped to a distant laboratory or if it is desired to make a weekly composite, the samples must be sterilized by the addition of chloroform, formaldehyde, mercuric chloride or some other germicide. Certain sterilizing agents interfere with certain analytical determinations, so that it is sometimes found desirable to sterilize two different portions of the sample, each with a different germicide. At Worcester. for example one set of samples is sterilized with formaldehyde and another with sulphuric acid.

The amount of the sterilizing agent required will depend upon the character of the sample. For strong sewage it may be necessary to use the equivalent of 1 to 2 c.c. of the concentrated chemical solution to each 500 c.c. of the sample, while for good sewage effluents and polluted waters one-half this amount of sterilizer may be ample.

Methods of Analysis.—A committee of the Laboratory Section of the American Public Health Association has given much study to the subject of proper methods of examination of water and sewage. The

methods adopted as a result of these investigations are published in "Standard Methods of Water Analysis," of which book a second edition was published 1912, and may be procured at the Association's office, 755 Boylston Street, Boston. In view of the critical study and final endorsement given to the methods outlined in this volume, it is highly desirable that they be followed generally by chemists, until superseded by better or more reliable ones in order that the results from different laboratories may be as nearly comparable as possible.

Expression of Results of Analyses.—In the United States the results of chemical analysis are usually expressed in parts per 1,000,000, which as ordinarily reported are equivalent to milligrams per liter. Formerly, and generally in Great Britain, such results were expressed in grains per gallon or in parts per 100,000, and these methods of notation are still in use at some laboratories. Table 1 will be found convenient for converting results expressed in one way into those expressed in another.

TABLE 1.—RELATIONS EXISTING BETWEEN GRAINS PER GALLON, PARTS PER 100,000, AND PARTS PER 1,000,000.

(From "Standard Methods of Water Analysis, 1913, p. 14)

	Grains per U.S. gal.	Grains per Imperial gal.	Parts per 100,000	Parts per 1,000,000
1 grain per U. S. gallon	1.000	1.20	1.71	17.1
1 grain per Imperial gallon	0.835	1.00	1.43	14.3
1 part per 100,000	0.585	0.70	1.00	10.0
1 part per 1,000,000	0.058	0.07	0.10	1.0

Parts per 1,000,000, strictly speaking, mean the weight of the specified substance present in 1,000,000 parts by weight of the water or sewage examined. As a matter of practice, however, the weights of the substances found are so small, relatively, that the specific gravity of the liquid is neglected or, more accurately, is assumed to be equal to that of pure water, unity, and a measured volume of the liquid is used for analysis.

As indicating the efficiency of a treatment plant, it is often the practice to record the "percentage removed" of various constituents. This term may be accurately applied in the case of certain ingredients, as suspended matter like silt; if 1 part is removed from 1,000,000 parts of sewage and 3 parts remain, it is correct to state that 25 per cent. of this suspended matter has been removed. If, however, instead of actual removal a substance has been merely altered in character, it may not be strictly accurate to refer to the changed condition in terms of "percentage removed." In order to avoid a misunderstanding it is preferable to use the term "percentage reduced."

In comparing the percentage reduction in the various constituents as determined at different treatment plants, it is important to take into consideration the character of the raw sewage and of the effluent. If a strong sewage is being treated, a relatively high percentage reduction in certain matters may be shown while at the same time the character of the effluent is inferior.

General Characteristics of Sewage.—In an article on "The Interpretation of a Sewage Analysis" by Prof. Earle B. Phelps (*Technology Quarterly*, vol. xviii), it is suggested that sewage possesses three general characteristics: the physical property of "concentration," the chemical property of "composition," and the biological property which he terms "condition."

Concentration is the term commonly used to designate the proportion of sewage matter to water in a given sewage, a strong or concentrated sewage containing a large proportion and a weak or dilute sewage a relatively small proportion of sewage matter. A strong sewage may be made weak or dilute by adding to it a sufficient quantity of water. In such a case the relative proportions of the several ingredients remain unchanged. A sewage may be strong on account of certain substances only. For example, the sewage from a city of 20,000 persons, which was normally weak or dilute, might be rendered strong by the admission of liquid wastes from industrial establishments. In this case the relative proportions of the several ingredients in the original sewage would probably be altered.

The composition of sewage refers more particularly to the relation between the different ingredients in it rather than to the degree of concentration. Thus a sewage containing tannery wastes may be high in fatty matters, while the sewage from a city containing iron works may contain large quantities of iron sulphate and free sulphuric acid.

The condition of the sewage is governed by the changes which have taken place in it. Sewage which has traveled but a short distance in a sewer is frequently alluded to as being "fresh," while that which has become putrid is referred to as "septic," terms discussed in more detail in the next chapter under "Progress of Decomposition with Diminution of Oxygen." No new ingredients have been added to the sewage, but a changed condition has been brought about which results, of course, in a change in composition. Such changes are due largely to biological action which will be discussed more fully in the next chapter.

Mineral and Organic Matter.—In a special report of the Massachusetts State Board of Health for 1890 entitled "Examination of Water Supplies," the late Dr. Thomas M. Drown, then chemist of the Board, gave the following description of the two main classes of substances, mineral and organic matter, in natural waters, which applies also to the matters found in sewage:

"In the examination of water we classify the substances found in it into mineral and organic. The distinction is not altogether a permanent one, for the mineral and organic are dependent on one another and, in part at least, pass into one another. From a sanitary standpoint our interest centers itself mainly in the organic matter. This we find, first, as living organisms, vegêtable and animal, which either float in the water or have the power to move about in it; second, the products of organic life, such as albumen, urea, tissue, etc., which may be dissolved in the water or suspended in it and third, products of the decomposition of organic matter. . . . . The ordinary processes of analysis suffice for the accurate determination of all the mineral constituents of the water, but the determination of the amount and character of the organic matter contained in water is not generally practicable, so that indirect methods must be resorted to to indicate its presence and condition. This difficulty is in part due to the very small quantity of organic matter usually present in natural waters and also to the rapidity with which it decomposes and loses its original character" (page 533).

The mineral, or inorganic, matter of sewage is usually of much less consequence than the organic matter. The fertilizing qualities, however, are due to the former class of substances, such as nitrogen, potash and phosphates, although these are combined to a greater or less extent with other elements in organic compounds. Sand, gravel and other mineral matters washed by storm water into sewers, and thence into ponds, rivers or harbors, may form deposits tending to obstruct navigation. Lime and certain other chemical substances from industrial wastes are inimical to fish, and acid iron liquors from galvanizing plants may cause the water into which sewage containing them is turned to become discolored and unsuitable for many industrial purposes. These conditions on the whole are exceptional. The chief dangers and troubles caused by sewage are due to its organic content.

Living Organisms or Bacteria.—Organic matter in sewage is composed largely of inanimate substance, but a portion is in the form of living organisms which may be said to be of two kinds: those capable of causing disorder and disease in the human system, and those upon which many of the processes of purification depend for the transformation of the objectionable organic substances into those of comparatively stable and inoffensive character. The latter may be properly termed "friendly" organisms, the former "unfriendly." Those thus classed as unfriendly apparently never become friendly. On the other hand, those which are classed as friendly may escape proper natural or artificial control and become decidedly unfriendly and cause a vast amount of trouble. The relation of living organisms or bacteria to the problem of sewage disposal is the subject of the next chapter.

Changes Taking Place in Sewage Matter.—Organic matter is made up of complex compounds of carbon, hydrogen, oxygen, nitrogen, sulphur

and other elementary substances. It is more or less readily broken up into other and usually simpler compounds, by chemical and biological action. Methods of chemical analysis are largely dependent upon such chemical action, while some sewage treatment processes are believed to be dependent to a large extent upon biological action. The latter, under conditions in which oxygen is absent, will cause changes in the organic matter involving the production of offensive odors, this process being known as "putrefaction," while under conditions of ample oxygen supply the resulting products will be either mineral or stable organic matter, this process being known as "oxidation." Both of these classes of matter, mineral and stable organic, are free from offensive odors and will not, under the ordinary conditions of nature, undergo changes resulting in the production of such odors. The problem of sewage treatment is primarily one of devising means and methods of utilizing these forces most effectively and economically in order to convert speedily the putrescible organic matter into mineral or stable organic matter without creating offensive conditions.

Organic matter possesses the interesting capability of passing from one condition through others in a cycle back to its original condition, which, for want of a better name, may be called the "cycle of life and death." In its living form it has organized structure, and it may or may not have the power of locomotion. Upon its death it is resolved, through the agency of living organisms, into simpler compounds, some of which are organic and some inorganic or mineral in nature. Some of these in turn serve as food for the living organisms, which use them for building up their structures, thus again incorporating them into the living body. This cycle is the foundation upon which many methods of sewage treatment are based.

Indestructibility of Matter.—At the outset the student of sewage disposal problems should understand that matter cannot be destroyed by any method of treatment. Substances may be changed in physical form, as sugar and salt are changed by solution in water, or they may be converted into several substances having entirely different properties and chemical composition, or they may be changed from solid form into gases; but whatever the changes may be there will be no loss of matter, and the aggregate weights of the parts into which a substance is divided will be equal to its original weight. Therefore, it follows that the objectionable characteristics of sewage must be overcome by removing the matter causing them and disposing of it apart from the sewage; by transforming it into unobjectionable matter; or by killing directly or indirectly the objectionable living organisms. Frequently all three methods are utilized in a single treatment plant.

Solids or Residue on Evaporation.—It is sometimes helpful to know how much total impurity, mineral and organic, water or sewage contains.

This is determined by evaporating a fair sample of known volume to dryness and drying at substantially the temperature of boiling water, this temperature being selected with a view to driving off all moisture and at the same time preventing the escape, volatilization, or conversion of substances which would be driven off at higher temperatures. Even under these conditions, all gases and small quantities of very volatile substances do escape and, therefore, are not recorded as forming a part of the contents of the sample. The error thus introduced is usually slight. The residue remaining after the water is evaporated is weighed and the results are reported in terms of parts per 1,000,000, from which can be computed the pounds of solid matter in 1,000,000 gal. of sewage, or other quantity as desired.

Suspended and Dissolved Solids.—The total residue on evaporation, obviously, includes both dissolved matter and that carried mechanically by the sewage, as sand or paper, some floating on the surface and some being held in suspension by the velocity of flow of the sewage. These two classes of solids are known as dissolved solids or solids in solution, and suspended solids or solids in suspension, respectively.

It is important to know the quantity of solids in suspension; in fact, usually this information is much more useful than the knowledge of either the total or the dissolved solids. To determine the proportions of the solids which are in solution and in suspension in sewage, it is first necessary to filter it, thus separating the two classes of substances. The filtrate, which contains only soluble matter, may be evaporated to dryness and the residue weighed, thus determining the quantity of substances originally dissolved in the sewage. By subtracting this result from the quantity of total solids obtained by evaporating an unfiltered sample, the quantity of suspended matter may be computed.

By another method, the solid matter collected upon the filter may be dried and weighed, thus directly determining the quantity of suspended matter, and the difference between this result and the quantity of total solids will give the quantity of matter in solution.

In some cases, it may be important to take into consideration the kind of filter used to separate the suspended matter when interpreting the results obtained. A dense filter may take out appreciably more suspended matter than one which is comparatively porous. Moreover, some sewages are of such a character that solids may precipitate during the act of filtration owing to colloidal matter, which is discussed on page 53. The results obtained, while apparently indicating a certain quantity of dissolved or suspended matter, may be far from the truth because of these changes.

Settling Solids.—The determination of solids in suspension in sewage is often valuable as showing how much can be, or has been, removed by different processes of treatment. For example, if sewage is passed

through sand filters, practically no suspended solids may be found in the effluent and a high percentage of removal is recorded. Where sewage is treated by processes of sedimentation, smaller proportions of the suspended matter are removed and the efficiency of the treatment is often recorded by noting the percentage removed. This, however, may be somewhat misleading, because by any practicable process of sedimentation it is not possible to remove all suspended matter. On this account, some engineers prefer to study the working of a sedimentation process by comparing the quantity removed in practice with that which can be removed when a sample of the sewage is allowed to stand quiescent in the laboratory for a period of time equivalent to that consumed by the passage of the sewage through the tanks, the latter quantity being referred to as settling or settleable solids.

Julius W. Bugbee, Chemist of the sewage treatment works at Providence, R. I., states that the settling solids are determined in his laboratory in the following manner:

"The samples for the determination of non-settling and settling solids . . . . are taken as follows: The 24-hour sample, brought in each morning, is well shaken and a portion proportional to the amount of the day's flow is measured out and placed in a bottle to form part of a sample for the week. After 4 hours another portion is taken from the 24-hour sample, this time by decantation and this is placed in another bottle for the weekly settled sample. Chloroform is used as a preservative in all samples."

The difference in solids in the two samples is assumed to be the quantity of suspended matter in the Providence sewage which is capable of settling in a period of 4 hours, which corresponds to the "detention period," or period of time consumed by the passage of the sewage through the tanks.

For such determinations, Dr. Karl Imhoff, Chief Engineer of the Sewerage Department of the Emschergenossenschaft, of Essen, Germany, has used the volumetric method for measuring suspended solids, the results being reported in terms of the volume of sediment, or sludge, accumulating in a predetermined period of time. To determine this he uses long tapering glasses about 4 in. in diameter at the top and about 17 in. long, of 1 liter capacity, the lower part being graduated into cubic centimeters and fractions (Fig. 1). A measured portion of the sewage as it enters the tanks is allowed to settle in one of the glasses for say 2 hours, and the volume of the suspended matter which collects at the bottom is compared with the volume similarly obtained from a settled portion of the effluent, in order to determine the percentage removal of the matter which is capable of settling in 2 hours.

The volumetric determination of settling solids by the conical glass method is open to serious errors under some conditions. Where the

sewage contains considerable coarse suspended matter, there is a tendency for the larger pieces to clog the narrow tapering portion of the glass, thus preventing the solids from filling the space below. This tendency is not ordinarily troublesome where effluent is being tested, owing to the finer nature of the suspended matter, the coarser particles having settled out in the tanks. The result of this error is that the apparent percentage removal of suspended matter is greater than the actual removal. After a time the volume of settling solids will begin to decrease, even while suspended matter is still depositing, owing to the gradual settling down or consolidation of the precipitate in the glass. This compacting of sludge is usually much more marked in the case of crude sewage than with an effluent, so that the percentage removal of suspended matter as indicated by readings of the conical glasses begins to decrease after a time and may be considerably less for 8 hours' sedi-

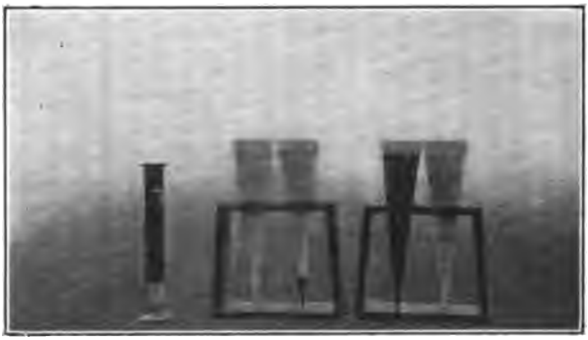

FIG. 1.—Conical glasses for determining the quantity of settling solids in sewages and effluents (after Imhoff).

mentation than for 2 hours. The variation in density of sewage and effluent sludges may make them hardly comparable. Another source of error is that in the case of various sewages and effluents, solids may be precipitated by contact with the inside of the glass, more or less of it settling to the bottom. The longer the samples stand, as a rule, the greater will be the error from this source.

A critical examination of the use of Imhoff glasses for determining settling solids in sewage at Worcester, Mass., has been made by R. S. Lanphear, Supervising Chemist of the Sewer Department, and the results of these tests are summarized in Table 33. It may be stated here that a correction made for the zero reading was often more than 50 per cent. of the corrected volume for 2 hours' sedimentation and in a large number of samples averaged over 25 per cent. Another correction was made by estimating the volume of the interstices between the sewage particles which were not filled with sludge. The total correction in some cases

amounted to over 100 per cent. of the true volume of settling solids. Still another estimate, also requiring careful judgment, was necessary because the surface of the sludge was rarely level, often sloping to such an extent that the maximum reading on one side of the glass was more than 33⅓ per cent. above the estimated measurement. After making all these corrections, duplicate tests gave remarkably uniform results. In spite of this fact, there was a wide variation on different days in the relation of the results of such determinations to those made gravimetrically, this ratio being more than three times as great in some cases as in others.

If this method is to be used for determining the efficiency of a sedimentation plant, great care should be exercised in interpreting the results, especially where the tests are made with sewages differing in quality and where the tests are comparatively few in number. It would seem that the most reliable service of the conical glass is to determine the volume of setting solids remaining in effluents, and in cases where the period of sedimentation is the same, such results may be fairly comparable. It is possible that the shape of the conical glasses now commonly in use may be modified in such a way as to make them more accurate in measuring the settling solids in crude sewage.

Colloidal Matter.—While for purposes of general classification the solid matter in sewage, as determined by the usual chemical analyses, is divided into matters in solution and in suspension, there is a portion known as colloidal matter which, strictly speaking, is neither in solution nor in suspension but rather in an intermediate state, from which it can be brought partly, but usually not wholly, into suspension.[1] Such substances are usually classed practically as matters in solution. The colloidal condition appears in some ways to be a transition stage, and it has been held by some that suspended matter will pass into this condition in the course of flowing a long distance in a sewer or in passing through pumps, and conversely, that contact in relatively quiescent condition with solid surfaces will throw matter out of the colloidal state into suspension. Thus such colloids are removed from sewage by sand filtration and accumulate in or upon the filtering material. Chemical changes like oxidation may precipitate some colloids.

As a practical matter, a substantial part of the solids usually reported as being in suspension when the determination is made by filtering through paper, evaporating and weighing, may be considered as being in a colloidal state. George W. Fuller states in "Sewage Disposal:"

"In sewage work the expression 'colloidal mater' is used rather loosely to mean those suspended matters in a state of very fine division which cannot

[1] Those substances which sometimes give to water a milky or turbid appearance may be classed as colloids, although colloids may be present in water without causing it to appear turbid.

be removed by sedimentation in practice. They exist apparently in a state of pseudo-solution or micro-suspension."

In his "Bacteriological and Enzyme Chemistry," Dr. Gilbert J. Fowler says in the opening paragraph:

"It was first shown by Graham that by appropriate means solutions could be obtained, which, while devoid of visible particles, were incapable of passing unchanged through a parchment membrane. Substances which were soluble and which would pass while in solution through a parchment membrane Graham termed crystalloids; substances which while soluble as judged by ordinary physical tests would yet not pass through a parchment membrane he termed colloids. A typical case illustrating the difference between a colloid and a crystalloid is the one selected by Graham, viz., silicate of soda. If a dilute solution of silicate of soda is carefully acidified with hydrochloric acid, no precipitation takes place; if the solution is now placed in a cylindrical vessel one end of which is closed by a parchment diaphragm and the whole immersed in clean water, which is renewed from time to time, the sodium chloride formed by the action of the hydrochloric acid on the sodium silicate will diffuse through the parchment and eventually be completely removed. The silicic acid will remain behind in the cylinder. The sodium chloride in this case is the crystalloid, the silicic acid the colloid. The apparatus used in the experiment is known as a dialyzer and the process as dialysis."

Fowler points out that no very definite line can be drawn between the two extremes of matter in the solid insoluble condition and matter in true crystalloid solution. He states that colloidal solutions are all found to contain particulate matter, that is, matter in an extremely divided state but still existing as separate particles. These may be seen by the use of the ultramicroscope, which depends upon the Tyndall ray, which lights up the particles as a sunbeam lights up particles of dust in the atmosphere.

Colloids can be precipitated from solution, usually by acidification, by the addition of solutions of various salts or by the introduction of other colloids, the last being illustrated by the precipitation of organic colloids by gelatinous mineral hydroxides, an action utilized in the clarification of sewage by chemical precipitation. True colloids conduct electricity very slightly if at all.

Practically speaking, then, the term "colloids" is often used in the discussion of problems relating to sewage treatment to include the very finely divided suspended matter which will not readily settle in ordinary sedimentation tanks and true colloid substances, which can hardly be classed as finely divided suspended matter in the sense that they are visible to the naked eye, but consist of substances which, under certain conditions, may be thrown out of their state of pseudo-solution and retained mechanically upon the surfaces of tank walls or of filtering materials.

There is no "standard" method of determining the quantity of colloidal matter in sewage and the attempt to determine and report it is rarely necessary, although it is probable that such matters are of much importance in the problem of sewage treatment and in some cases, especially where industrial wastes are treated, careful consideration must be given to the effect of such matters upon the devices employed in treatment. It is important also, in some cases, to bear in mind the existence of such substances in the selection of methods of analyses as well as in their interpretation.

Fixed Residue, or Mineral Matter.—After having evaporated a sample of sewage to dryness and determined the weight of the total solid matter, chemists sometimes ignite the residue at a relatively low temperature to burn off organic matter. The weight of the matter remaining represents approximately the quantity of mineral matter usually reported as "fixed residue" or "fixed solids." This method of treatment may volatilize some inorganic compounds and other changes may take place in the mineral matter, which tend to make this determination a rather unreliable index to the quantity of such matter.

The dissolved fixed solids determined from the filtered sample represent the dissolved mineral matter, and the difference between the total and the dissolved fixed residue represents the suspended mineral matter.

Loss on Ignition, or Organic Matter.—The difference between the weights of the fixed residue and of the total residue represents approximately the quantity of organic matter in the original sample, which was burned off. This result is affected by the volatilization and chemical changes going on in the inorganic matter, as already described, and thus is not the exact quantity of organic matter present. It may be useful, however, in some cases, particularly where applied to sewage or water containing a relatively large proportion of organic matter or a small proportion of volatile mineral salts or chemical substances changed by heating to the temperature of ignition. The determination of loss on ignition, however, simply gives a general idea of the quantity of organic matter present, and indicates nothing as to its quality or how it will behave under artificial or natural conditions.

The dissolved loss on ignition is similarly determined by igniting the residue resulting from the evaporation of the portion of the original sample which was filtered for the determination of the total dissolved solids.

The suspended loss on ignition is calculated by subtracting the dissolved from the total loss on ignition.

Nitrogen Cycle.—The nitrogen cycle is the term applied to the transformation of nitrogen compounds, by which the nitrogen first found combined with other elements in organic compounds in the living

organism, is later found combined in dead organic matter, then in different inorganic combinations as a result of varying degrees of oxidation, and finally again in organic combination in living organisms. This cycle has been an important factor in the study of problems relating to the treatment of sewage and is given more detailed attention in the next chapter.

Protoplasm, an essential constituent of living organisms, contains a class of substances known as proteids, in which carbon, hydrogen, nitrogen and oxygen are found and sometimes sulphur and phosphorous. They are known also as albumins or proteins. Many foods consumed by man and animals contain large quantities of albumin, which is found in considerable proportions in peas and beans as well as in eggs and meat. Thus comparatively large quantities of nitrogen are taken into the system, a portion of which goes to make up the animal tissue and a portion is secreted with other waste products. Plant life consumes large quantities of nitrogen, one of the principal active ingredients of commercial fertilizer, and this is incorporated into the structure of the plant.

Nitrogenous organic matter consumed by animals and absorbed by plants undergoes changes within the organisms. Nitrogen compounds find their way into the sewage in the form of excreta from animals, animal and vegetable wastes from the kitchen and other wastes from the household and industrial establishments. The nitrogen compounds in the sewage are present in many forms, among which albumin and urea are typical.

Sewage contains relatively large quantities of albumin, the behavior of which during processes of decomposition has received much study. Ordinary white of egg contains albumin known as egg albumin, which is typical of many such substances in sewage. Fowler states that egg albumin is composed of carbon, 50 to 55 per cent.; hydrogen, 6.9 to 7.3 per cent.; nitrogen, 15 to 19 per cent.; oxygen, 19 to 24 per cent.; sulphur, 0.3 to 2.4 per cent. It is a colloidal substance precipitated by aluminum and ferric hydroxide.

As discharged into the sewers, some of the organic nitrogen compounds are relatively stable and others only loosely bound together. Some of the nitrogen is present even in inorganic form, as ammonia and ammonium compounds. The organic compounds are easily broken up directly or indirectly by bacterial action into other substances, and the nitrogen, some of which was at first in highly complex, relatively stable substances, becomes part of less complex and less stable matters, while what at first was loosely bound has been liberated from its organic combination and may be present in some inorganic form as ammonia, or it may have been liberated as nitrogen gas.

In the treatment of sewage by bacterial action, much depends upon

the conditions under which the processes are carried on. Sometimes the action is conducted in the absence of air, as in an air-tight or air-trapped tank, and the changes are confined to an interchange of the elementary chemical substances present in the sewage. This process is often termed hydrolysis, because it generally is accompanied by a breaking up of the molecules of water and the combination of the hydrogen and oxygen of the water with carbon, nitrogen and other substances from the organic matter. In this way the organic compounds may yield free nitrogen, N, in a gaseous state, ammonia, NH_3, consisting of nitrogen combined with hydrogen, carbon dixoide, CO_2, a combination of carbon and oxygen, and methan, CH_4, another gas consisting of carbon and hydrogen.

In other cases, the bacterial action goes on in the presence of an abundant supply of oxygen, supplied either directly from the atmosphere or from that dissolved in relatively pure water. Under such circumstances the action is one of oxidation, the organic substances being split up and combining with oxygen, the nitrogen finally appearing as nitrates of some base or alkali, as sodium nitrite ($NaNO_2$) and sodium nitrate ($NaNO_3$). These two processes are not as simple as might be inferred from this brief description and are explained more fully in the next chapter.

The organic nitrogenous compounds, after transformation into ammonium compounds, nitrites or nitrates, become suitable food for plants. When an effluent is turned into water, this mineral nitrogenous matter becomes available to nourish microscopic plants and these, in turn, become food for fishes and other animal life. Thus the nitrogen cycle from life to life is completed.

There are accurate, convenient methods for determining the quantity of nitrogen present in sewage and effluent and of ascertaining whether it is in relatively stable or unstable combination as organic matter, or in any one of three combinations indicating progressive change from the organic substance to the highly oxidized mineral compound, nitrates. It is this fact, that by determining the character of the nitrogen compounds present in the sewage or effluent it is possible to ascertain the extent to which oxidation of the organic matter has proceeded, coupled with the fact that the nitrogen determinations can be readily and accurately made, which has given them such importance and resulted in their wide adoption, rather than any particular significance in the quantity of nitrogen present. As a matter of fact it is quite probable, as has been pointed out by Dr. Wm. P. Dunbar, that if specific information is desired as to the quantity of trouble-making matter present, the determination of organic sulphur might throw more light upon the problem than the determination of nitrogen. While too much weight may have been given to nitrogen determinations and their re-

sults may have been misinterpreted in some cases, the fact remains that they indicate with considerable accuracy the condition of the organic matter present in sewage or effluent. The determinations of organic nitrogen in most municipal sewages serve as valuable indices of its strength in organic matter. But all nitrogen determinations should be considered as broad measures, rather than as indices of specific quantities of a definite substance. It is, therefore, important to give the correct interpretation to such results, particularly because they not only indicate the quantities in a general way, but also the character and condition of the organic substances, their history, what changes they have undergone and what changes they may yet undergo or induce.

Organic Nitrogen.[1]—Organic nitrogen is the nitrogen combined with carbon and other elements in the form of organic matter. In municipal sewages not much affected by industrial wastes or by hard water from the water supply or from ground-water infiltration into the sewers, the ratio of the nitrogen to carbon is fairly constant, and it is to be expected that the nitrogen-sulphur ratio is equally so. Even in a sewage containing large quantities of sulphates from the water supply or ground water, this relation will be fairly constant from day to day, subject, of course, to the fluctuations in the proportion of infiltration. The total quantity of nitrogen is determined by prolonged boiling with strong acid, and finally distilling the ammonia thus formed. This treatment is severe enough to break up the more stable organic nitrogenous compounds and to liberate the nitrogen in the form of free ammonia. By determining the total organic nitrogen in a filtered sample, information is obtained concerning the quantity of dissolved nitrogenous organic matter, and, by difference, the quantity of suspended nitrogenous organic matter.

Albuminoid Ammonia.—Organic nitrogen compounds in sewage may be divided into two main classes, those which are readily converted into ammonia by boiling with alkaline permanganate of potash for a short time, and those which are not so decomposed. The latter class is sometimes called residual organic nitrogen. The nitrogen given off from the less stable portion of the nitrogenous organic matter is combined with hydrogen in the form of ammonia, NH_3. This is distilled off and accurately measured, the results usually being reported "nitrogen as albuminoid ammonia." This name is given to it because the nitrogen of albumin may be driven off in this form by the same treatment. The dissolved albuminoid ammonia is determined in a filtered sample and that suspended is computed by difference. Albuminoid ammonia does not exist in sewage as such, but it is an index

[1] Sometimes and perhaps more properly called "Kjeldahl Nitrogen" because it is determined by a method known to chemists as the "Kjeldahl Method;" as reported, it does not include nitrogen as free ammonia, nitrites and nitrates.

of the less stable nitrogenous organic compounds, and is taken as a measure of their quantity. If, as is sometimes the case, the results of this determination are reported as albuminoid ammonia they may be expressed in terms of nitrogen by multiplying the amount of albuminoid ammonia by the factor $^{14}/_{17} = 0.82$.

There is no sharp dividing line between substances yielding so-called albuminoid ammonia and those more stable matters not readily yielding their nitrogen under this treatment. In fact, upon prolonged treatment by this method, sewage will continue to yield nitrogen, and if carried far enough a large part, or perhaps all, of the nitrogen of the more stable substances, will eventually be given off. The quantity of albuminoid ammonia in sewages and other liquids, as ordinarily determined by this method, is very variable, ranging roughly from 20 to 40 per cent. of the total organic nitrogen content, and on this account the "Standard Methods of Water Analysis," of the American Public Health Association states that the albuminoid ammonia results are less valuable than those of the total organic nitrogen, and recommends for sewage, effluents and highly polluted streams, that albuminoid ammonia determinations be omitted and that in their place the total organic nitrogen be determined. The authors have used both determinations for many years and believe that a knowledge of the quantity of nitrogen as albuminoid ammonia in sewage and corresponding effluents may be of considerable value to the operator of treatment plants, and that in many cases it is well worth determining, if, indeed, it is not worth more than the determination of total organic nitrogen. In studying sewages from different cities or the condition of sewage from the same city sampled at different times, it is often desirable to determine both the nitrogen as albuminoid ammonia and the total organic nitrogen. By deducting the former some idea may be formed of the relative quantity of the more stable forms.

Free Ammonia.—When perfectly fresh, sewage contains some free ammonia. As has already been explained, free ammonia is an intermediate product in the nitrogen cycle. It is formed, under certain conditions, in several processes of sewage treatment, and is important as indicating to some extent the condition of the biological process. For many years it has been determined by distillation, but this method is open to some uncertainty, because some of the unstable substances, like urea, give off their nitrogen upon heating to the extent required for distillation. For this reason it has been suggested that free ammonia be determined directly in the sample, without distillation. This is practicable in many cases, and where the results are reliable, this method is to be preferred.

As free ammonia is a decomposition product, the quantity present is a valuable indication of the freshness of the sewage. The fluctuation in the quantity of free ammonia in the effluent from some treat-

ment processes is a helpful guide to the efficiency of the biological action upon which the treatment depends. Free ammonia is inorganic in nature, and while it may or may not indicate an unfavorable condition of the treatment plant, it does not of itself constitute a source of offensive odor, and it does not ordinarily represent organic matter which will decompose and create offensive conditions. Practically all free or saline ammonia is in solution and ordinarily no effort is made to determine separately the small quantity which may be in suspension.

Nitrites and Nitrates.—The conversion of ammonium compounds formed by oxidation, as mentioned on page 56, into nitrites and finally nitrates is called nitrification, and is the last step in the complete biological purification of sewage. The determination of nitrogen in the form of nitrites and of nitrates is, therefore, instructive as regards the efficiency of such processes. The results are usually reported in terms of nitrogen, although the oxygen combined in these compounds, being available for further oxidation, is of considerable importance in imparting stability to an effluent.

The quantity of nitrites is determined by adding to the sample quantities of acetic acid solutions of sulphanilic acid and *a*-naphthylamine, which produce a pink color if nitrites are present. The depth of color thus produced is compared with that of a set of standards containing progressively increasing quantities of sodium nitrite and treated in the same manner. The sample contains the same quantity of nitrites as the standard which it matches.

Nitrates may be determined by a colorimetric method similar to that used for measuring the nitrites. The sample after evaporation is treated with phenolsulphonic acid and made slightly alkaline. If nitrates are present a yellow color is produced, the depth of which, when compared with similarly treated standard solutions of potassium nitrate of known strength, indicates the quantity of nitrates in the sample. This method is not applicable to waters containing more than 30 parts per million of chlorine, as is sometimes the case with sewages and often with industrial wastes. Such samples should be treated by the reduction method, by which the nitrates are reduced to ammonia by nascent hydrogen generated from strips of aluminum foil placed in the samples after they have been made alkaline with potassium hydrate. After reduction the ammonia is distilled off and measured as in the determination of free ammonia.

Oxygen Consumed.—Determinations of nitrogen do not give any information as to the quantity of organic matter in which nitrogen is absent. As a measure of the carbonaceous as well as the nitrogenous organic matter, the so-called determination of "oxygen consumed," "oxygen absorbed" or "oxygen required" has been long used. As implied in the name, the sample is so treated that its organic matter is oxidized in

such a manner that the quantity of oxygen combining with the organic constituents can be readily computed. To do this the sample of sewage or effluent is accurately measured, acidulated and treated with a known quantity of a solution of potassium permanganate of standard strength. It is then placed in a bath of boiling water and digested for exactly 30 minutes, according to the standard method of the American Public Health Association. The practice in different laboratories varies considerably, however. In some the sample is acidulated and then brought to the boiling point, at which time the potassium permanganate is added and the whole digested for 2, 3 or 5 minutes, or such other period as may be preferred. In England this determination is usually carried out at room temperature, and observations are taken of the quantity of permanganate used up at the end of 3 and 15 minutes and 4 hours. One advantage of the method of the Association over common boiling methods lies in the fact that oxygen absorbed by volatile organic compounds is included, whereas in samples brought to boiling before the permanganate is added, these substances are driven off and their effect in absorbing oxygen is not noted.

The quantity of the oxidizing agent remaining after boiling, or digesting at lower temperature is measured, and the quantity of oxygen used up or "absorbed" by the sample is computed. This process is, in a way, like the loss on ignition, except that it is wet instead of dry combustion, and is carried out in such a manner that the quantity of oxygen absorbed is under control and subject to accurate measurement.

This determination is frequently regarded as relating to the carbonaceous matter, but the nitrogenous matter containing carbon is also oxidized, so that in reality this determination, in so far as it goes, is a measure of both nitrogenous and non-nitrogenous organic matter, while the nitrogen determinations affect only nitrogenous matter.

Unfortunately, the combustion is never complete, so that the quantity of oxygen absorbed is only that used up by the more easily attacked substances, and the quantity consumed depends largely upon the length of time the digestion is continued. Thus many different results may be obtained from the treatment of the same sample for different periods of time, and different sewages may yield widely different results with the same treatment, depending upon the proportion of the readily oxidizable substances contained in them. This determination is, therefore, of most service when used for comparing the quality of sewages from the same community from day to day, or comparing a sewage with the corresponding effluent, using any of the several available methods of treatment.

Substances most readily subject to putrefaction appear most easily oxidized, and consequently the oxygen-consumed tests generally indicate the most objectionable portion of the sewage, from the point of view of its decomposition and putrefaction. It follows that a smaller oxy-

gen-consumed content of an effluent may indicate a change of the corresponding quantity of the more putrescible substances in the original sewage. This conclusion is generally justified, but to reason that the organic matter indicated by the oxygen-consumed test of an effluent is in its original form and has the same characteristics may not be at all justified. For example, the charred embers of a burned log contain carbon as did the original log, but it has lost its original shape, color, organization and many of its properties. They are still organic matter, but of a very different character. In the same way the effluent from a filter bed will contain some organic matter; its solid contents will lose weight on ignition, and it will consume oxygen when boiled with a strong oxidizing agent, but the organic matter may be entirely different in nature and character from that of the sewage.

The results of the oxygen-consumed determination are not ordinarily reported so as to give an idea of the weight of organic matter in a given volume of sewage or effluent, but rather in terms of the quantity of oxygen required for its combustion, and are thus only of relative value, furnishing a means of comparison. In other words, the oxygen consumed is only a comparative, indirect indicator of the concentration of the sample with respect to organic matter.

The total carbon may be obtained by evaporating the sample to dryness, taking precautions to prevent the loss of carbon by volatilization of its compounds, and determining the total carbon by combustion in a special form of apparatus. This process is long and tedious, and its use in the analysis of sewage and water would not ordinarily be justified.

In the United States the consumed oxygen is usually determined both in the unfiltered and filtered samples, thus giving the "total" and the "dissolved" oxygen consumed, the difference being the "suspended" oxygen consumed. In Germany, according to Dunbar, the sample is usually filtered before being treated with the oxidizing agent.

The chief value of this determination, aside from enabling one to form an idea of the concentration or strength of the sewage, is to determine the amount of work accomplished by a treatment process, the difference between the results obtained by treating the sewage and the effluent representing the organic matter removed or changed by the process of treatment. Dunbar says:

"If the oxygen absorbed of the crude sewage is known, and this is compared with that of the effluent, conclusions may be drawn as to the putrescibility of the effluent. In 1899, as a result of numerous experiments, I was able to state that domestic sewage is deprived of its putrescible character if purification is carried out to such an extent as to reduce the oxygen absorbed, estimated by Kubel's method, by 60 to 65 per cent. Experience gained since then has only served to strengthen this view." ("Principles of Sewage Treatment," page 251.)

It would be dangerous to accept this statement by Dunbar as applicable generally to the proportion of the organic matter which must be removed from sewage in general to render it non-putrescible, on account of the great variation in the quality of different sewages. Many of them will hardly be non-putrescible when the oxygen consumed has been reduced 60 or 65 per cent.

Comparison of Oxygen-consumed Values Determined by Different Methods.—In order to compare the results of oxygen-consumed tests made by different methods, it is necessary to know the relation existing between the values obtained by the various methods. To reduce the results as nearly as possible to a uniform basis, Table 2 and Fig. 2 have been prepared, using the data published in the report of the

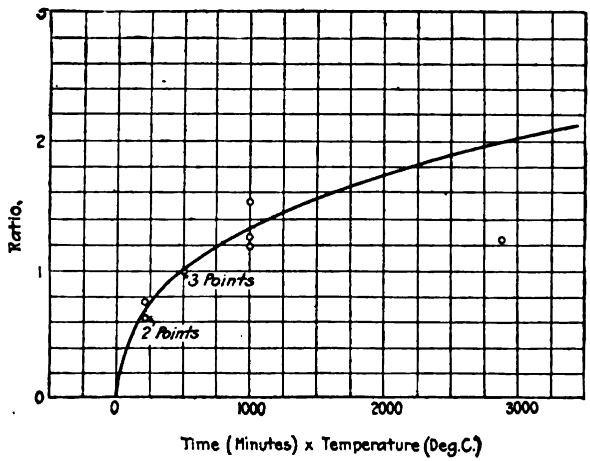

FIG. 2.—Ratio of values of oxygen consumed (as determined by digesting at or near 100°C. for the several contact periods in use) to the value determined by the 5-minute period.

Columbus experiments (page 49) and the results of comparative tests at the Lawrence Experiment Station (Report Mass. St. Bd. Health, 1905, page 365). Table 3 furnishes a means of readily converting the results obtained by one method into those that would have been obtained by any other. The authors realize that the data upon which these tables and diagram are based are meager, and that the 30-minute ratios vary too widely to be of accurate application. Nevertheless it is believed that these ratios may prove helpful in the absence of more satisfactory means of conversion.

Putrescibility Tests.—The consumption of oxygen, the basis of the oxygen-consumed tests, is due strictly to chemical action; that is, the acid permanganate of potash is a sufficiently strong oxidizing agent to

break up organic compounds, the carbon thus liberated combining with the oxygen of the permanganate. In nature this same combination of carbon and oxygen takes place, but from entirely different causes, being

TABLE 2.—RATIOS OF VALUES OF OXYGEN CONSUMED AS DETERMINED BY DIGESTING, FOR THE SEVERAL CONTACT PERIODS IN USE, TO THE VALUE DETERMINED BY THE 5-MINUTE (BOILING) PERIOD

Time of contact (minutes)	Temperature, degrees Cent.	Relative results to 5-minute boiling; authority					
		Mass. State Bd. of Health	Columbus Exp.	Fuller	Kinnicutt	Worcester*	Average
† 2 (a)	100.0	0.75	0.74	0.65	0.65	0.89	0.70
† 5 (b)	100.0	1.00	1.00	1.00	1.00	1.00	1.00
‡ 5 (c)	100.0	....	....	....	....	1.09	1.10
‡10 (d)	100.0	....	1.18	1.25	1.52	1.16	1.32
‡30 (e)	96.0	1.50	3.14	....	....	1.06	2.00§
240 (f)	100.0	....	4.73	4.00	....	....	4.36
3 (g)	26.7	....	0.20	0.20	....	....	0.20
15 (h)	26.7	....	0.33	0.35	0.30	....	0.33
240 (i)	26.7	....	0.56	0.60	0.48	....	0.55

* Worcester not included in average because of unusual character of sewage.
† Permanganate added after sample is heated to boiling.
‡ Permanganate added before heating sample.
(a) Used at Worcester, Mass., and at Lawrence, Mass., prior to 1905.
(b) Used at Gloversville, N. Y., and Lawrence, Mass., after 1905.
(c) Used at Columbus Experiment Station, 1905.
(d) Method commonly used in Germany.
(e) Standard of American Public Health Association, used at Boston, Mass. (Massachusetts Institute of Technology Experiment Station), and Philadelphia, Pa.
(f) Special test used at Columbus Experiment Station.
(g), (h), (i) Methods commonly used in England.
§ Weighted average.

largely the work of living organisms which feed upon the organic matter. In the physiological process the original substances are broken up and the carbon and oxygen united in a stable combination, substantially unattended by disagreeable odors. The oxygen required to oxidize the dissolved and suspended matters is derived from the supply dissolved in the water, including the oxygen combined with nitrogen in the form of nitrates and nitrites. If there is an insufficient supply of this oxygen, an entirely different biological process will set in, due like that of oxidation to the work of living organisms, but of a different kind. The organic matter will be broken up as in the other process, but the products of this process will contain among them offensive smelling matter, such as hydrogen sulphide, H_2S, which has the odor of a rotten egg.

Dunbar attributes much value to the determination of the organic sulphur in sewage and effluents, and states:

"It is the reduction product of this organic sulphur, sulphureted hydrogen, which causes putrefying sewage to be a nuisance. . . . The organic nitrogen of decomposing sewage is not indicated by any particular odor, although it is in the case of putrefying concentrated urine." ("Sewage Treatment," page 37.)

TABLE 3.—CONVERSION TABLE FOR OXYGEN CONSUMED BY DIFFERENT METHODS; BASED ON CURVE OF FIG. 2.

	2 min. at 100°C.[1]	5 min. at 100°C.[1]	5 min. at 100°C.[2]	10 min. at 100°C.[2]	30 min. at 96°C.[2]	240 min. at 100°C.	3 min. at 26.7°C.	15 min. at 26.7°C.	240 min. at 26.7°C.
2 min. at 100°C.[1]	1.00	1.43	1.57	1.89	2.86	6.23	0.29	0.47	0.79
5 min. at 100°C.[1]	0.70	1.00	1.10	1.32	2.00	4.36	0.20	0.38	0.55
5 min. at 100°C.[2]	0.64	0.91	1.00	1.20	1.82	3.96	0.18	0.30	0.50
10 min. at 100°C.[2]	0.53	0.76	0.83	1.00	1.51	3.30	0.15	0.25	0.42
30 min. at 96°C.[2]	0.35	0.50	0.55	0.66	1.00	2.18	0.10	0.17	0.28
240 min. at 100°C	0.16	0.23	0.25	0.30	0.46	1.00	0.05	0.08	0.13
3 min. at 26.7°C	3.50	5.00	5.50	6.60	10.00	21.80	1.00	1.65	2.75
15 min. at 26.7°C	2.12	3.03	3.33	4.00	6.06	13.21	0.61	1.00	1.67
240 min. at 26.7°C	1.27	1.82	2.00	2.40	3.64	7.93	0.36	0.60	1.00

[1] Permanganate added after sample is heated to boiling. [2] Permanganate added before heating sample. The table is read as follows: results of boiling 2 minutes at 100°C., adding permanganate to the boiling sample, are 0.70 of those after boiling 5 minutes; results of boiling 30 minutes at 96°C., permanganate being added before boiling, are 10 times those after boiling 3 minutes at 26.7°C.

Hydrogen sulphide and other similar substances are not formed in the presence of an abundant supply of oxygen.

The oxidation of organic matter in water requires considerable time, much more at 0° to 15°C. than at 15° to 30°C., and in a general way, within the ordinary natural limits, the rapidity of the action is proportional to the temperature, being practically *nil* at 0°C. If oxidation is to predominate, there must be enough oxygen to combine with the organic matter whenever required. If the sample contains such supply, it is said to be non-putrescible, for it will not putrefy and give off offensive odors.

One familiar with a given treatment plant can usually ascertain if its effluent is putrescible by the ordinary analyses, knowing from the quantity of albuminoid ammonia or oxygen consumed, and nitrates, whether the treatment has gone sufficiently far to produce an effluent which will not putrefy.

As oxygen is required to prevent putrefaction and as water can dissolve only a limited quantity of oxygen, an effort has been made to determine

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the stability or putrescibility of samples by determining the quantity of oxygen dissolved in them after standing for different periods of time, adding thereto the oxygen available from nitrates and nitrites, although the quantity which can be derived from the latter is often negligible. By making such determinations and observing the loss in oxygen due to the changes going on in the sample, a knowledge of its putrescibility may be obtained. If the oxygen is not all used up the sample will prove stable. Others have suggested the determination of putrescibility by ascertaining whether the sample turns black or produces offensive odors, which, if they occur, indicate a putrescible effluent.

Samples subjected to the latter tests are kept in completely filled, tightly stoppered glass bottles at room temperature during the period of observation.

The standard putrescibility test of the American Public Health Association consists in collecting a sample in a 150 to 200-cc. glass-stoppered bottle. To this a small quantity of methylene blue is added. The sample is then incubated, preferably at 20°C. during 4 days, observations of the color being made at least once each day. If the sample is stable or non-putrescible, the blue color of the chemical will be retained. If, on the other hand, the sample is putrescible, the color will disappear, on account of the reducing action of sulphur compounds.[1] This test is based upon the action of organisms contained in the sample. If there is sufficient organic matter of the right kind to maintain their existence in adequate numbers, they will exhaust the supply of oxygen and the action will be changed from oxidation to putrefaction, in which case the reducing sulphur compounds will be formed. If, on the other hand, the quantity and quality of the organic matter are not suitable for the rapid growth of the organisms, there will be an ample supply of oxygen to meet the requirements of such changes as go on in the organic matter, and the process will be one of oxidation throughout and will not be attended by the production of offensive odors.

Table 4 gives the relative stability numbers corresponding to the time of incubation at both 20° and 37°C. The relative stability number is assumed to indicate the ratio of available oxygen in the sample to that required for the complete oxidation of the organic matter. For example, if a sample of effluent when incubated at 20°C. retains its color only until the end of the third day, it is said to have a stability number of 50, which means that it contained, as dissolved oxygen and oxygen in nitrates and nitrites, one-half of the oxygen required for complete oxidation of the organic matter.

The "Standard Methods of Water Analysis," recognizing that this test is often more severe than the requirements under natural condi-

[1] The test is not applicable to effluents disinfected with hypochlorites, if any of the disinfectant remains in the sample, on account of the action of the hypochlorites.

tions, stipulate in effect that a sample which retains its color for 4 days when incubated at 20°C. may be considered as practically stable "except when great accuracy is desired." This arbitrary assumption reduces the labor involved in making tests and is undoubtedly justified in many cases. If it is desired to obtain the results of this test quickly, the sample may be incubated at 37°C., for the time required for decolorizing at this temperature is but half that required at 20°C. The authors have felt that the additional information derived by continuing the incubation for 14 days at 20°C. was well worth the labor involved.

The methylene blue test requires only simple, inexpensive apparatus, and can be carried out by persons without knowledge of chemistry. It has been quite generally adopted as a routine test for the control of sewage treatment and dilution. Where merely treatment to insure freedom from offensive conditions is needed, it will be found a useful test and perhaps, in some cases, the only one required.

TABLE 4.—RELATIVE STABILITY NUMBERS

("Standard Methods of Water Analysis," page 64)

t_{20}	t_{37}	S	t_{20}	t_{37}	S
0.5	....	11	8.0	4.0	84
1.0	0.5	21	9.0	4.5	87
1.5	....	30	10.0	5.0	90
2.0	1.0	37	11.0	5.5	92
2.5	....	44	12.0	6.0	94
3.0	1.5	50	13.0	6.5	95
4.0	2.0	60	14.0	7.0	96
5.0	2.5	68	16.0	8.0	97
6.0	3.0	75	18.0	9.0	98
7.0	3.5	80	20.0	10.0	99

S = Relative stability or ratio of available oxygen to oxygen required for equilibrium; expressed in per cent.

t_{20} = Time in days to decolorize methylene blue at 20°C.

t_{37} = Time to decolorize at 37°C.

Theoretical relation:

$S = 100\ (1 - 0.794^{t_{20}})$.

$S = 100\ (1 - 0.630^{t_{37}})$.

Oxygen Requirements of Sewage and Polluted Waters.—Where the object of treating sewage or industrial wastes is merely to prevent the production of foul odors in natural waters, it is only necessary to produce an effluent which shall not deposit any considerable quantity of putrescible organic matter and which shall require no more oxygen for its further purification than the available oxygen supply in the stream or body of water into which it is turned. It is, therefore, important in

such cases to ascertain the oxygen requirements of the sewage or effluent in question.

In the case of some particular dilution projects this may be determined by diluting the sewage or effluent with known volumes of the water available for dilution and submitting the diluted samples to the methylene blue test. The oxygen requirements are then expressed in terms of the number of volumes of diluting water required to maintain stability for the desired length of time. Obviously such tests should be made with the water available for dilution since the organic matter contained in this water and its dissolved oxygen content are factors influencing its diluting capacity.

The oxygen requirement expressed in terms of oxygen used up in a stated period of time is a measure of the putrescible matter in sewages, effluents and polluted waters. The Royal Commission on Sewage Disposal held that this test provides the most trustworthy chemical index of the probable effect of sewage or effluent upon a stream. The English method of making the test, which is there called the dissolved oxygen absorption test, is described in the Appendix to the Eighth Report of the Commission, page 93. Briefly, it consists in furnishing the sample under examination with a proper volume of aerated tap water and incubating the diluted sample for 5 days at 65°F. (18.3°C.). The difference between the dissolved oxygen in the initial diluted sample and in the sample after incubation gives the oxygen required by the amount of the original sample used for dilution.

C. B. Hoover introduced a similar test, which he called the dissolved oxygen-consumed test, as a regular procedure at the Columbus sewage works, to determine the efficiency of the treatment. His method is described in *Engineering News*, May 28, 1914. The samples, after dilution, are incubated at 37°C. for 24 hours.

To avoid some of the cumbersome features of the dilution method and possible errors due to a reduction in concentration of bacteria and bacterial food supply, Dr. Arthur Lederer has suggested that the "biochemical oxygen demand" be provided for by adding to several portions of the sample under examination progressively increasing known quantities of sodium nitrate. From a series of tests he found:

"A definite amount of atmospheric dissolved oxygen resulted in an improvement (in the polluted water) equivalent to the addition of the same amount of saltpeter (nitrate) oxygen, as judged by the methylene blue putrescibility test." (*Journal of Infectious Diseases*, vol. xiv, page 482.)

The oxygen requirements in this case are computed from the loss in available oxygen in the form of dissolved oxygen, nitrates and nitrites after incubation for the desired length of time, although they can be estimated roughly as follows: (1) by submitting the nitrate-treated

portions to the methylene-blue test, noting the decolorized sample which contained the largest quantity of nitrates; (2) by allowing the portions to stand in full, stoppered bottles at a uniform temperature, as 20°C., and noting the portion which first shows evidence of putrefaction by turning black or evolving offensive odors; (3) by ascertaining the lowest concentration in which nitrates can be detected by colorimetric methods.

The above methods afford certain definite information which the older methods of analysis do not give and it is possible that in some cases the oxygen requirement determined in some such way may supplant the ordinary determinations of organic matter.

Dissolved Oxygen.—It is important not to confuse the determination of dissolved oxygen with that reported as oxygen "consumed," "required" or "absorbed." The last three are the technical names of the process of chemical combustion of organic matter, described on page 60, and are the terms in which the quantity of organic matter is expressed. The results reported as dissolved oxygen are obtained by the determination made to ascertain the quantity of atmospheric oxygen which is dissolved in the given sample of water.

The atmosphere contains about 79.0 per cent. nitrogen, 20.9 per cent. oxygen and 0.1 per cent. of other substances, including carbonic acid gas, which in very pure air amounts to about 0.022 per cent. and in the air of cities about 0.045 per cent. These gases are soluble in water, and, being constantly in contact with natural bodies of water, are always present in them but in proportions different from those of the air. Water freely shaken with air will dissolve 65.1 volumes of nitrogen and 34.9 volumes of oxygen. (*Jour. N. E. W. W. Assoc.*, vol. iii, page 37.) Ground waters are not in such intimate contact with the air and, therefore, frequently contain less dissolved oxygen than surface water, and some artesian waters are devoid of oxygen.

Pure water constantly in contact with the atmosphere is capable of dissolving only a definite quantity of oxygen at any temperature and atmospheric pressure. This quantity varies inversely with the temperature, as can be seen by Table 5. The oxygen may be expelled from water by boiling.

Pure water dissolves more oxygen than does impure water. Therefore, soft surface water is capable of dissolving more oxygen than hard surface water or sea water. The greater the salinity of the sea water or the lower the dilution of normal sea water with surface water, the less oxygen it is capable of dissolving.

The quantity of oxygen which a given sample of water can normally dissolve varies with the barometric pressure. Pure water at sea level will dissolve 9.17 parts of oxygen per 1,000,000 parts of water at 68°F. At an altitude of 5000 ft. it will dissolve only 7.59 parts at the same tem-

Table 5.—Solubility of Oxygen in Fresh Water and in Sea Water of Stated Degrees of Salinity at Various Temperatures when Exposed to an Atmosphere Containing 20.9 Per Cent. of Oxygen and under a Pressure of 760 Mm.

(Adapted from Standard Methods for the Examination of Water and Sewage, American Public Health Association, Second Edition, page 62)

Temperature		Chlorine in sea water (parts per million)					Difference per 100 parts of chlorine per million
Fahr.	Cent.	0	5,000	10,000	15,000	20,000	
		Milligrams per liter					
32.0	0	14.62	13.79	12.97	12.14	11.32	0.0165
33.8	1	14.23	13.41	12.61	11.82	11.03	0.0160
35.6	2	13.84	13.05	12.28	11.52	10.76	0.0154
37.4	3	13.48	12.72	11.98	11.24	10.50	0.0149
39.2	4	13.13	12.41	11.69	10.97	10.25	0.0144
41.0	5	12.80	11.09	11.39	10.70	10.01	0.0140
42.8	6	12.48	11.79	11.12	10.45	9.78	0.0135
44.6	7	12.17	11.51	10.85	10.21	9.57	0.0130
46.4	8	11.87	11.24	10.61	9.98	9.36	0.0125
48.2	9	11.59	10.97	10.36	9.76	9.17	0.0121
50.0	10	11.33	10.73	10.13	9.55	8.98	0.0118
51.8	11	11.08	10.49	9.92	9.35	8.80	0.0114
53.6	12	10.83	10.28	9.72	9.17	8.62	0.0110
55.4	13	10.60	10.05	9.52	8.98	8.46	0.0107
57.2	14	10.37	9.85	9.32	8.80	8.30	0.0104
59.0	15	10.15	9.65	9.15	8.63	8.14	0.0100
60.8	16	9.95	9.46	8.96	8.47	7.99	0.0098
62.6	17	9.74	9.26	8.78	8.30	7.84	0.0095
64.4	18	9.54	9.07	8.62	8.15	7.70	0.0092
66.2	19	9.35	8.89	8.45	8.00	7.56	0.0089
68.0	20	9.17	8.73	8.30	7.86	7.42	0.0088
69.8	21	8.99	8.57	8.14	7.71	7.28	0.0086
71.6	22	8.83	8.42	7.99	7.57	7.14	0.0085
73.4	23	8.68	8.27	7.85	7.43	7.00	0.0083
75.2	24	8.53	8.12	7.71	7.30	6.87	0.0083
77.0	25	8.38	7.96	7.56	7.15	6.74	0.0082
78.8	26	8.22	7.81	7.42	7.02	6.61	0.0080
80.6	27	8.07	7.67	7.28	6.88	6.49	0.0079
82.4	28	7.92	7.53	7.14	6.75	6.37	0.0078
84.2	29	7.77	7.39	7.00	6.62	6.25	0.0076
86.0	30	7.63	7.25	6.86	6.49	6.13	0.0075

perature, 17 per cent. less than at sea level. Ordinarily, the differences in altitude between different sampling points are so small that it is unnecessary to correct results for them and it is therefore customary to report results as though the barometer stood at 760 mm. or 29.92 in.

Water is said to be saturated with dissolved oxygen when it contains all, and no more than, the quantity which it is capable of dissolving when intimately mixed with air at the specified temperature and pressure. Waters usually contain less than this amount and the results of dissolved-oxygen tests are reported in percentages of saturation, as 60 per cent. saturation or 60 per cent. of the saturation value. As temperatures fluctuate greatly and likewise the oxygen content of the water, it is necessary to make such comparison with the saturation value at the same temperatures. To facilitate such comparisons, reference may be had to Table 5.

The third column of this table gives the saturation values of pure distilled water, while the other values apply to sea water of different degrees of salinity. It is important in determining the dissolved oxygen in sea water to ascertain also its salinity.

If water has remained in contact with air long enough under more than normal pressure and the pressure is then suddenly released, it will at such instant contain more than its saturation value of dissolved oxygen and may be said to be super-saturated. When a sample of cool water saturated or nearly saturated with oxygen is allowed to stand in a warm room bubbles of oxygen may be observed forming in the water and collecting on the sides of the container. At such a time the water may become super-saturated and some of the dissolved oxygen will escape. This condition is due to the fact that the warmer the water the less oxygen it will hold. By agitating the water the excess oxygen will escape and by boiling the water, all dissolved oxygen may be driven off. Under some conditions in nature water may become super-saturated with oxygen liberated by certain microscopic organisms, as in the Massachusetts stream receiving sand filter effluents, described in Chapter IV.

The results of the determination of dissolved oxygen are reported in terms of milligrams per liter, which is equivalent to parts per million, in cubic centimeters per liter or in percentage saturation.

Because the saturation value of water varies so greatly with different conditions of composition, temperature and pressure, it is always wise to consider the results in terms of milligrams or cubic centimeters per liter

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Note to Table 5.—Under any other barometric pressure, B, the solubility can be obtained from the corresponding value in the table by the formula:

$$S' = S\frac{B}{760} = S\frac{B'}{29.92}$$

S' = Solubility at B mm. = B' in.
S = Solubility at B mm. = 760 mm. = 29.29 in.

To convert milligrams per liter or parts per million into cubic centimeters per liter, multiply by 0.7; to convert cubic centimeters per liter into milligrams per liter, multiply by 1.43; weight of 1 liter of oxygen at 0°C. and 760 mm. pressure equals 1.43 grams.

that the actual quantity of oxygen present may be appreciated and one may not be misled by the percentage of saturation, which is often a convenient expression, although it may be the same in two samples in which the total quantities of oxygen are materially different. For example, if the oxygen has been used up by biological processes in two waters at temperatures of 15° and 27°C. respectively, until the percentage of saturation has fallen to 50 the impression might be gained that the same quantity of oxygen had been used up and also that the same quantity remained in the waters. At these temperatures, however, as will be seen from Table 5 the dissolved oxygen remaining is 5.07 and 4.03 parts per 1,000,000 respectively. The water at the lower temperature contains about 25 per cent. more oxygen than the other in spite of the fact that its loss of oxygen has been 25 per cent. greater.

Chlorine.—Common salt (sodium chloride, $NaCl$) is present in all natural waters, usually being more plentiful in waters near the seacoast than in those inland. It is also an ingredient of food and is present in large quantities in kitchen refuse, wash waters, urine and feces. Salt is, therefore, found in sewage in much larger quantities than in most natural waters, and its presence in the latter in excess of that normal to them, indicates probable contamination by sewage. As the chlorine combined with the sodium in salt may be easily and accurately determined, it is customary to analyze the water for this only, disregarding the sodium and reporting the result in terms of chlorine.

Much work has been done to determine the quantity of chlorine normal to natural waters in different localities. After making such determinations at a great number of places in a given area, the figures are sometimes platted upon a map of the area, and lines called isochlors drawn through the points having equal chlorine content. The State Board of Health of Massachusetts, in 1890, published such a map of Massachusetts, from which the normal chlorine can be ascertained for any locality within reasonable limits of accuracy.

Salt is not decomposed by the changes occurring in sewage, either natural or artificial, and for this reason its determination is often important, as indicating whether or not samples of effluent and sewage correspond; if they do the quantity of chlorine should be substantially the same in both.

In a general way, the quantity of chlorine per inhabitant reaching the sewers in a given unit of time is uniform; therefore the chlorine content of the sewage may be used to determine its strength or concentration, by which is meant the proportion of sewage matter in a given volume of water. For this determination it is necessary to know the quantity of chlorine in the water supply, and also any possible source of an unusually large quantity of chlorine discharged into the sewers. In a similar way the quantity of infiltration into the sewers may sometimes be ascertained

by the degree of dilution afforded the sewage, the chlorine content of the ground water being known. Effluents from irrigation fields and intermittent sand filters often contain large quantities of ground or surface water, the proportion of which to sewage effluent may be ascertained by a study of the relation of the quantity of chlorine in the effluent to that in the sewage applied.

Chlorine is practically all in solution in sewage or effluent so that it is not necessary to make determinations in both filtered and unfiltered samples.

Fats.—Fats in sewage come from kitchen refuse; from packing-house, wool-scouring, tannery and other industrial wastes; from used soap and many other sources. They form an element of value in sewage and efforts have been made to devise processes for their commercially successful recovery. They are very resistant to bacterial action and tend to form a coating over bacterial surfaces, thus interfering with the work of filters. It is important to determine occasionally their quantity in sewage and in sludge, particularly in the latter because of the important effect of the fats in diminishing the value of sludge as a fertilizer, discussed in Chapter XVII.

One result of the presence of fats in sewage is the oil or oily sleek often seen floating on the surface of sewage polluted waters. The sewage field about a sewer outlet in sea water is disclosed by such a film of oil, or "sleek," upon the surface.

Iron.—The determination of iron in sewage is of importance only in the exceptional cases where industrial wastes introduce large quantities of this element into the sewers. The determination of iron in effluents may be of value as indicating the condition of sand filters. Relatively large quantities of dissolved iron compounds characterize effluents from beds having tendencies toward putrefaction. Such a condition produces a reducing action, rendering the ferruginous compounds in the sand soluble in water.

Sulphur.—Sulphur in sewage may have its origin in animal matter, in the wastes from industrial establishments, in the ground water, or in the water supply. It is important as one of the substances most likely to yield offensive odors under putrefactive conditions. In many cases its determination, directly or indirectly, is of considerable value, as the quantity present may account for phenomena observed or to be expected.

The sulphur in animal tissues is combined with carbon in organic matter; in industrial wastes it may be present as sulphites, sulphates or sulphides, and in ground water and water supplies, as sulphates. In the decomposition of the sewage, the form of the sulphur compounds is usually changed, and it is frequently necessary to determine the sulphur in the resultant forms.

As sulphur is responsible for many offensive odors during putrefac-

tion, its presence is indicated by putrescibility tests. The blackening of the samples is usually due to the formation in them of sulphides, generally sulphide of iron.

Phosphates.—Small quantities of phosphorus compounds are found in sewage and they constitute one of its elements of fertilizing value. A determination of these compounds is rarely made in sewage analysis, but may be important in studies of the fertilizing value of sewage or sludge.

Potash.—Other fertilizing ingredients of sewage are potassium salts, which are soluble in water and do not ordinarily exist in appreciable quantities in sludges. In studies of the fertilizing value of sewage it is desirable to determine the potash, but this is unusual in ordinary sewage investigations. In connection with problems of disposal of some industrial wastes, such as wool-scouring liquors, potash may be an important consideration.

Alkalinity and Acidity.—Some sewage contains alkalies such as ammonium, sodium, and potassium carbonates and bicarbonates and similar salts of the alkaline earths, calcium and magnesium. The authors have seen samples of sewage so strong that fumes of ammonia rose from them into the air and samples in a barrel evolved so much ammonia that when hydrochloric acid was exposed near the surface of the sewage, the upper portion of the barrel was filled with the characteristic white fumes of ammonium chloride.

Sewage is normally alkaline due to the alkalinity of the water supply, the ground-water infiltration and much of the sewage matter itself. When testing to determine the degree of alkalinity, it is not usual to ascertain by analysis what alkaline salts are present but simply to determine the alkalinity from all sources and report the same in parts per million of calcium carbonate. Excessive alkalinity is inimical to both plant and animal life, so that this consideration may be an important one in connection with the discharge of certain industrial wastes into natural waters.

Sewage which contains mine drainage, wastes from wire drawing plants and some other industrial wastes will often be found acid. The results of such tests do not usually show to what acids or salts the acidity is due but simply the total acidity in terms of sulphuric acid in parts per million parts of sewage. The degree of acidity has an important bearing upon the inauguration and maintenance of bacterial activity within a sewage filter. Acid exerts an inhibiting action upon bacteria and it is possible to carry the acidity to the point where it will kill all bacterial life.

Gases.—Sewage generally contains minute quantities of gas. Organic suspended matter, by the processes of nature and by some methods of treatment, yields gases as decomposition products and their determina-

tion may be of importance in some instances. Such decomposition may result in the formation of methane, commonly found in the mud of marshes and hence called marsh gas; hydrogen sulphide, due to the breaking up of organic sulphur-bearing compounds or the splitting off of oxygen from sulphates by reduction; carbon dioxide, formed by the breaking up of carbonaceous matter and the combination of the carbon with oxygen; nitrogen, liberated in gaseous form by decomposition of nitrogenous organic matter; and small quantities of other gases. Routine sewage analysis does not include the determination of gases, which are made only in special investigations.

Appearance.—As a part of the chemical analysis, observations are frequently made of the turbidity, sediment and color of samples of effluent and water into which sewage and effluents have been discharged. Usually such observations are not made on sewage although helpful in special cases.

In making turbidity observations of polluted waters it is not ordinarily necessary to go to the refinement required with potable water. It is generally sufficient to designate the turbidity as "slight," "distinct," "decided," "milky," etc. Only in rare cases is it necessary to record the sediment, and when necessary it should be reported as "slight," "distinct," or "decided."

The color of effluents or waters into which sewage and effluents are discharged is not ordinarily significant. In unusual cases, however, due to the discharge of industrial wastes, it may be important to note the color in general terms.

Odor.—Observations of the odor of water containing sewage or effluent, or of an effluent itself, may furnish valuable information, but this is not usually the case with sewage. Water in a stream may appear quite free from odor, when observed in the usual way, but if it is examined in a bottle, a very different impression may be gained.

For such an examination a large bottle, preferably one holding 2 to 4 qt., should be half filled with the water, the stopper replaced, the bottle vigorously shaken for about 1 minute, and then the odor observed by quickly placing the nose to the mouth of the bottle. Some samples which yield only a slight odor when cold, will give off a decided odor upon heating, due to either of two causes; the odor may be distributed through the water in such a way that it is not expelled readily upon shaking, but is driven off upon heating, or the water may contain organic matter, from which odoriferous compounds are distilled upon heating. Where odors are caused by gases dissolved in the sample, they are sometimes so completely liberated by shaking the sample, that the second shaking will not produce further odor. Therefore, in cases of doubt, another sample should be tested. Where the observed odor is caused by organic substances, it will usually persist and may change in quantity

and character as the sample is kept, due either to multiplication of organisms, as in cases of surface waters, or to decay of such organisms or of other organic matter, as in the case of sewage.

Drown stated in the report of the Massachusetts State Board of Health upon Examination of Water Supplies, 1890:

> "Except in cases where the odor is very faint it does not require any special apparatus, skill, or experience to detect an odor in water; hence the judgment of one is as good as another in this respect. But, in drawing inferences as to the origin and significance of an odor, the judgment may go very far astray" (page 567).

In a general way, the authors' experience coincides with Drown's statement, although they consider that experience makes the perception keener so that slight differences are noted by the trained observer which escape the observation of other persons. Furthermore, they have found that different types of odors are found in different cases; for example, the odors observed in effluents from intermittent sand filters are usually quite different from those of effluents from trickling filters, and odors observed in streams into which sewage or effluent has been discharged may differ from either.

For many years, the Massachusetts State Board of Health reported its observations of odor of samples of river water and effluent from sewage filters, and for this purpose adopted such terms as "faint," "musty," "distinctly musty," "decidedly musty," "disagreeable," "decidedly disagreeable," and "offensive." Drown stated:

> "The odor that comes from sewage contamination in water is very different from that class of odors which we have been considering (odors from organisms). A stream badly polluted by sewage . . . . has more or less the odor of sewage itself. In more dilute condition the odor is musty, both cold and hot; this is quite characteristic. But it must not be supposed that the sense of smell furnishes us with a delicate test for the detection of sewage contamination. When we perceive this mustiness even faintly, the water is generally badly polluted, considered from the standpoint of a drinking water." ("Examination of Water Supplies," page 569.)

While it is difficult to convey to others the exact meaning of arbitrary terms adopted for designating the character and strength of odors the authors have found the observation of odors helpful, particularly to the operator of filters of types commonly used. Many years ago they adopted, in connection with the operation of a sand filter plant, a scale of odors to indicate the efficiency and condition of the filters. This scale, provided with both numbers and descriptive terms, was as follows: 0, none; 1, slightly musty; 2, distinctly musty; 3, decidedly musty; 4, offensive; 5, very offensive.

This scale of odors was used in recording daily observations of effluent

of intermittent sand filters at Worcester. With these observations, there were parallel tests of the putrescibility of the effluent with methylene blue and by observing whether the samples standing in a full tightly sealed bottle gave offensive odors or became blackened. An experienced observer could tell by the odor within narrow limits, whether or not the samples were putrescible. Such samples as had no odor, or but a slight musty odor, were non-putrescible without exception. Such samples as were observed to be 3 or higher on the scale were putrescible almost without exception, while samples observed as 2 on the scale were generally non-putrescible.

It is believed that such a scale of odors may be used at other sewage plants to enable the operator to judge the quality of the effluent immediately upon taking the sample without waiting for the results of chemical analyses. The odor of effluents from intermittent sand filters may vary in different places, according to the character of the sewage or wastes applied to them. The effluent from a filter receiving paper mill wastes, for example, has a very different odor from that from filters receiving ordinary municipal sewage. A scale of odors may be adopted, however, for the filter receiving paper mill wastes which will give the operator a good working knowledge of the condition of his filter and the stability of his effluent. An experienced observer can also often gain valuable information in this way as to the condition of the filter from which the effluent has come. For example, the formation of black layers in sand filters is generally accompanied by a characteristic odor in the effluent from them.

Organic Growths.—Much can often be learned with respect to the character of an effluent or a polluted water by observing the nature of the organic growths which are found at the effluent outlets, along the banks and bed of a stream or in the water itself. Such growths constitute the plankton which is the subject of Chapter IV.

CHAPTER III

BACTERIA AND THEIR RELATION TO THE PROBLEM OF SEWAGE DISPOSAL

Among the living things which play a part in the disposal of sewage are rats, the scavengers of sewers, in which many live; gulls and other birds which feed upon floating organic matter discharged from sewers; fish, which often congregate about sewer outlets for food; certain plants which thrive on some nitrogen compounds of sewage; and micro-organisms,[1] plants and animals too small to be seen without the aid of magnifying glass or microscope. This last class appears to be the most important.

The microscopic organisms or plankton, which are the subject of Chapter IV, bring about some of the changes in the cycle of life and death, but the resolution of complex organic matter into simpler forms and its oxidation to stable organic and mineral substances appear to be due largely to bacteria. Furthermore, the transmission of disease by sewage appears due to bacteria.

Sewage contains enormous numbers of both harmful and helpful bacteria. The former cause illness and perhaps death after entering the human body, or create disagreeable conditions in waters receiving relatively large quantities of sewage. If sewage causes offense, certain bacteria are responsible; if it is oxidized to odorless substances possessing fertilizing value, certain other bacteria are the cause. Chemical analyses show what changes have been wrought in sewage by the treatment,

[1] Prof. Wm. T. Sedgwick suggested that the micro-organisms be divided into two classes, given in Prof. Whipple's "Microscopy of Drinking Water," third edition, page 10, as follows:

Micro-organisms Organisms, either plants or animals, invisible or barely visible to the naked eye.	*Microscopic Organisms (Plankton)* Not requiring special culture. Easily studied with the microscope. Microscopic in size, or slightly larger. Plants or animals. *Bacterial Organisms* Requiring special cultures. Difficultly studied with the microscope. Microscopic or submicroscopic in size. Plants.

Brevity favors the term "plankton" instead of "microscopic organisms," with which it is practically synonymous when applied to those plants and animals generally considered in sanitary engineering. Plankton was the term applied by Victor Hensen, 1887, to all minute plants and animals that float free in the water. Plants attached to the shore and animals that possess strong powers of locomotion were not included in the plankton, but fragments of shore plants, fish-eggs, young fish-fry, and the like were included, according to Whipple.

but they do not reveal the living cause of these changes, upon which successful sewage disposal depends. Chemical analysis of polluted water discloses its general character but not the presence or absence of disease germs. Where sewage has contaminated water, the resulting ill effects upon the health of drinkers of the water have been due, so far as demonstrated, to disease-producing bacteria. It is evident, therefore, that a study of bacteria is essential to a correct understanding of the sewage disposal problem.

BACTERIA AND METHODS OF STUDYING THEM

Although ancient philosophers imagined invisible minute organisms might cause some of the visible changes in matter,[1] bacteria were first seen by Anton van Leeuwenhoek about 1683. Nobody else paid attention to the subject until about a century later, when several investigators took it up. Finally Dr. Louis Pasteur began his researches which showed what a tremendous influence bacteria exert on living conditions and many industries. The methods of investigation were difficult and their results often indefinite until Prof. Robert Koch discovered in 1882 a means of isolating single species of bacteria. With this discovery, bacteriology became a science of great importance.

Morphology[2] of Bacteria.—Most known species of bacteria have a rodlike shape, whence their generic name, bacteria (Greek, bakterion, a staff). Other forms exist, however, as shown in Fig. 3, from Schenk's "Elements of Bacteriology." The rod bacilli average about 2 μ in length and 0.5 μ in diameter.[3] ("General Bacteriology," Dr. Edwin O. Jordan, fourth edition, page 54.) The bacillus of typhoid fever is, Jordan states, 1 to 3 μ in length, and the bacillus of influenza averages only 0.5 μ. The weight of a typical bacillus, 1 μ square in cross-section and 2 μ long, with a specific gravity of 1, was calculated by Dr. Otto Rahn as 0.000,000,002 mg. (Marshall's "Microbiology," page 88.) Of this weight he estimates at least four-fifths to be water.

Bacteria multiply by fission, or a division of the cell into two practically equal portions. Under favorable conditions they multiply with great rapidity, which accounts in part for the large amount of work accomplished by them. Fortunately, during the disintegration of their food substances, acids and other products injurious to them are commonly formed, which accumulate until further multiplication is stopped. Checks to rapid development are also offered by an insufficient food

[1] "Like precautions must be taken against swampy places for the same reasons and particularly because as they dry, swamps breed certain animalculae which cannot be seen with the eyes and which we breathe through the nose and mouth into the body, where they cause grave maladies." (Varro's "Rerum Rusticarum," B.C. 36, Fairfax Harrison's translation in "Roman Farm Management.")

[2] Science of form and structure of organisms.

[3] 1μ = micron or micro-millimeter = 0.001 mm. = about 1/25,000 in.

supply, lack of moisture, unsuitable temperature, and the competition of other species of bacteria. Another feature of the life of some species, particularly the bacilli, is the spore, described by Jordan as follows:

"Physiologically the spore is to be considered as a resting stage. It serves to tide the species over a period of dryness, famine, or unsuitable temperature, and to preserve alive in a hostile environment a sufficient number of individuals until such time as favorable conditions recur. The spore stage is,

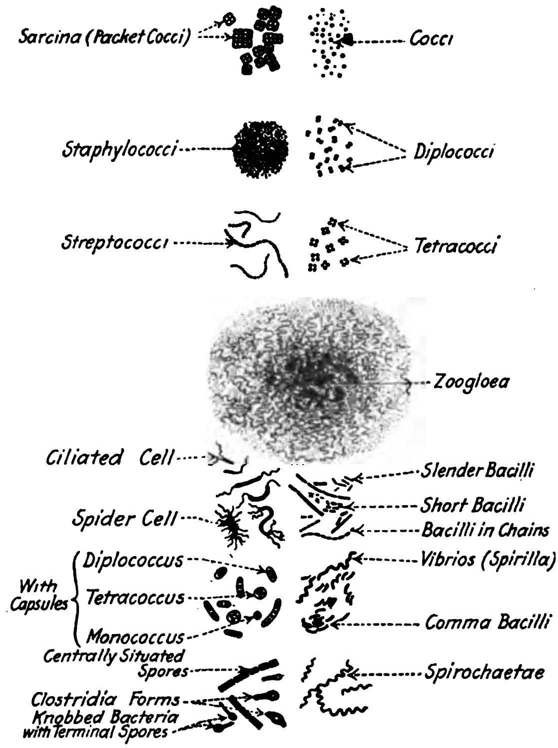


FIG. 3.—Forms of bacteria (after Baumgarten). (From Schenck's "Elements of Bacteriology," page 3.)

in fact, physiologically analogous to the periods of hibernation or estivation among higher forms of life. In this resting state the living matter of the spore may remain dormant for years or even for decades." ("General Bacteriology," page 67.)

"The spore of the anthrax bacillus (Fig. 4) when brought under favorable conditions, shows first a change in the refractive property of the spore substance; this is followed by a slight elongation of the spore, with a final bursting through of the spore membrane and the outgrowth of a short rod,

which then divides in the usual manner. The new outgrowth of the anthrax bacillus takes place at the pole of the spore; in the closely related hay bacillus it is at the equator. Other forms of bacteria exhibit intermediate methods of germination, and irregularities sometimes occur in the development of spores of the same species." (*Ibid.*, page 66.)

Classification of Bacteria.—Bacteria are sometimes classified into saprophytes and parasites. The former organisms live on dead matter and the latter on the substance of living bodies. Obligate parasites include species capable of living only upon animate matter, and the facultative parasites are species able to exist upon either dead or living matter. Another classification, the most useful in sewage treament work, is based upon the need of oxygen, and was explained on page 10.

FIG. 4.—Spores, bacillus of symptomatic anthrax. Methylene-blue stain (Kolle and Wassermann). (From Jordan's "General Bacteriology," page 66.)

To acquire a knowledge of their forms, motility, manner of multiplication and spore formation, it is necessary to study bacteria alive under the high-power microscope. The determination of species identified by their physical characteristics and ability to absorb certain stains, is made by drying cultures of the bacteria under investigation upon glass cover-slips and then staining them with various anilin dyes, when they can be readily examined under the microscope. These methods are not as yet ordinarily used in studying sewage disposal problems.

Culture Media.—Much of our knowledge of bacteria has been obtained by observing their growth under different artificial conditions and their effects upon other living organisms, and by examination of the products of their life processes. While a single bacterium is invisible to the naked eye, it will multiply within a few days under favorable circumstances into a colony so large that the mass may be seen easily without a

magnifying glass. It was early found that many bacteria multiply rapidly in meat extract containing a little peptone. This forms the basis of many of the culture media in common use. When gelatine is added to the broth, the mixture will form at ordinary room temperature a transparent amber-colored jelly, which holds the bacteria in position while each grows into a colony, an invaluable discovery by Koch.

Nutrient gelatine melts at about 25°C.,[1] and certain species of bacteria produce substances which liquefy it. To overcome these difficulties, a vegetable gelatine called agar is substituted for the ordinary gelatine. It forms a jelly which melts at about 40°C., making possible its use at blood temperature, 37°C., and bacteria do not secrete substances which liquefy it.

Modifications of these media are made by adding relatively small quantities of lactose and saccharose to the broth and nutrient gelatine, and dextrose or lactose and litmus to agar. Other media are milk and litmus milk used to detect bacteria producing coagulating enzymes or acids, and blood-serum and other substances on which certain species will grow and others will not. In the former case they produce products which identify the bacterium under observation. Certain species of bacteria produce gas of a practically uniform quality when growing in nutrient broth of proper composition. Some bacteria produce acid substances while others closely allied to them do not. In Chapter II the disappearance of color from sewage or effluent samples treated with methylene blue, page 66, was mentioned as a method of determining the putrescibility of the sample. This is due to a reducing (deoxygenating) action by bacteria which have so multiplied in the sewage culture medium that the oxygen has been exhausted and hydrogen sulphide formed.

Effect of Temperature.—Few if any bacteria are capable of multiplying at 0°C., and most bacteria in the vegetative stage are killed by exposure at a temperature of 100°C. Jordan states:

"Three temperature limits may be distinguished: a minimum, or the lowest point at which growth occurs; an optimum, or the temperature of most luxuriant growth; and a maximum, or the highest temperature at which growth can take place. The position of these three points differs greatly for different species." ("General Bacteriology," page 70.)

Sewage disposal is influenced by the great reduction in bacterial activity as the temperature falls toward 0°C. This reduced activity

[1] °F. = $\frac{9}{5}$°C + 32.
°C. = $\frac{5}{9}$(°F − 32).
20°C. = 68°F. = room temperature.
25°C. = 77°F.
37°C. = 98.6°F. = blood temperature.
40°C. = 104°F.
100°C. = 212°F. = temperature of boiling water.

results in less danger in using ice taken from a contaminated[1] water than in drinking the same water.

It has been found by investigation that most of the typhoid bacteria frozen in ice die at once, although a few may live even 6 months.

The thermal death-point is the temperature at which a species of bacteria is killed upon exposure for a given period of time, as 10 minutes. Jordan gives the thermal death-point (10 minutes exposure) for the cholera spirillum and the typhod bacillus as 58° to 60°C. He states further:

"It may be noted that the thermal death-point of those bacteria that are at all likely to be present in polluted water is low, 57° to 60°C., and since these micro-organisms do not form spores, the practice of simply bringing the water to the boiling-point suffices to insure its safety for drinking purposes." ("General Bacteriology," page 72.)

Sterilization[2] of Apparatus and Culture Media.—Bacteria are practically omnipresent, and on this account, as well as because they are invisible, the prevention of bacterial contamination of a substance under investigation requires the use of sterilized apparatus and culture media. Apparatus uninjured by high temperatures may be sterilized by exposing it at a temperature of 170°C. for 45 to 60 minutes, and other apparatus by prolonged boiling. Some instruments are best sterilized by heating them in an open gas flame. Culture media may be sterilized in the autoclave by exposure to steam under pressure, at a temperature of about 120°C. (248°F.). Heating culture media in test-tubes at this temperature for 5 minutes, and those in large containers for 15 minutes, is sufficient. Some media must be sterilized at lower temperatures. Most bacteria in the vegetative stage are killed by boiling, although

[1] *Contamination*, the introduction into a water of bacteria or other substances which tend to render it unsuitable for domestic use. The degree of contamination may increase from merely nominal, according to the quantity and nature of the contaminating substances, until it reaches a maximum, when the water is unsuitable for domestic consumption in its present condition and cannot be made so by practicable methods of treatment. In general, contamination refers to the introduction of such substances as will tend to render the use of the water dangerous through the possible presence of pathogenic organisms. It is not possible to define, according to contents as determined by analysis, the line on the one side of which a water may be said to be contaminated and on the other polluted, and local conditions will have an effect in determining whether a water should be designated as contaminated or polluted. It is even conceivable that a water might fairly be classed as contaminated in one locality and polluted in another, although such a water would in the one case be very highly contaminated and in the other slightly polluted.

Pollution, the introduction into a water of substances of such character and in such quantity that they tend to render the body of water or river objectionable in appearance and to cause it to give off objectionable odors. The degree of pollution may increase from merely nominal, when a water has just reached a degree of contamination which renders it unfit for use or for preparation for use, as a domestic supply, until it becomes obviously filthy and offensive. In general, pollution refers to the introduction of organic matter which, under the conditions, produces an objectionable appearance or which, due to biological changes, may cause the water to become objectionable in appearance or to give off offensive odors.

[2] By sterilisation is meant the killing of all bacteria, including their spores, on the apparatus or in the material treated.

some spores are not destroyed when boiled even for a number of hours; by 2 or 3 exposures at a temperature of 100°C. for 15 or 20 minutes at a time, sterilization can be accomplished. Between exposures, the medium is kept at 20°C. for 24 hours, to enable the spores to develop into the vegetative stage. Media which cannot be heated to 100°C. are sterilized by exposure in the autoclave at lower temperatures for longer periods of time, and on a greater number of successive days.

Method of Counting Bacteria.—There is no method known of ascertaining the exact number of bacteria in a liquid. The number is estimated by counting the number in a very small portion of the sample and assuming that each equal unit volume of the water contains an equal number. It is impracticable to determine with a microscope the number of bacteria even in a drop of polluted water. It is therefore customary to encourage each bacterium to multiply independently of the others, until it has formed a mass or colony of bacteria large enough to be seen with the naked eye or with a low power lens.

This is accomplished by mixing 1 cc. of the sample, diluted with sterile water if necessary, with about 10 cc. of nutrient gelatine or agar heated to liquefaction. This mixture is spread evenly over the bottom of a glass Petri dish, about 10 cm. in diameter with vertical sides about 1 cm. high, so that the culture medium will be of even thickness at all points. The dish has a glass or porous earthenware cover, of the same shape and slightly greater diameter, to exclude air bacteria from the gelatine. The plate is cooled on a level surface until the gelatine congeals, and is then transferred to an incubator in which a temperature of either 20° or 37°C. is usually maintained, depending upon the character of the investigation.

The quantity of the original sample mixed with the medium is so regulated that the total number of bacteria upon the plate shall not exceed 200. By mixing the sample with ten times its volume of medium, the bacteria are practically isolated one from another and are so held after the medium has congealed. After 1 to 3 days the colonies will be large enough to be counted, and in this way the number of bacteria ascertained, as each colony stands for one bacterium. Such counts do not give the total number of bacteria, because the obligate anaerobes cannot multiply under the conditions ordinarily provided in such studies and certain nitrifying organisms and many other bacteria are unable to grow on either gelatine or agar plates. In many sewage investigations this fact is not of controlling importance, for the numbers of such organisms are relatively small and the errors are not sufficiently large to effect the results materially.

Counting Excessive Numbers of Bacteria.—In some waters, and especially in sewage, the smallest quantity which it is practicable to take for examination, 1 cc., contains so many bacteria that it would be impossible to count the colonies on the plates. Where the number in 1 cc. is likely

Fig. 5.—Effect of dilution of sample on number of bacterial colonies.

to cause more than 200 colonies on a plate, 1 cc. of the sample may be mixed with 9 cc. of sterilized distilled water, and 1 cc. of this diluted sample plated. Many times even this dilution is insufficient, and 1 cc. of the original sample is diluted with 99 cc. of the sterilized water. This dilution is continued until bacteria on a plate will not exceed the number which it is practicable to grow and count.

Fig. 5 illustrates the results of such dilution. Plate A is from 1 cc. of sample plated without dilution. The number of colonies, about 600, was too large for easy and accurate counting. Plate B is from a dilution of 1 to 9. This plate contained about 60 colonies and could be counted with ease and accuracy. Plate C is from a dilution of 1 to 99, which proved unnecessarily great. Bacterial counts are reported in terms of the number of bacteria per cubic centimeter of the water examined.

Identification of Species.—Great labor has been expended in isolating different species of bacteria and describing them so as to make their future identification practicable. This is of great importance, for while much can be learned by a study of a mixture of organisms, most progress will be made when the action of the several species is known and the environments required for the optimum development of each are defined. Then it may be possible to produce conditions encouraging the rapid multiplication of the bacteria desired and at the same time keeping down other growths. Under such conditions, much more and better work is to be expected of the bacteria present.

BACTERIA IN SEWAGE

Bacteria are to be expected in sewage, for it consists largely of the bacteria-carrying water supply of the community. Some also doubtless fall into the sewage from the air, the wastes of some industries contain large numbers, and many more are washed into sewers by storm water. By far the greatest number, however, come from the excreta of man and animal, which teem with bacteria.

Number of Bacteria in Natural Waters.—The number of bacteria found in natural surface waters varies greatly, according to the character of the drainage area. Streams from steep and rocky mountain sides have a low bacterial content. Streams draining cultivated fields or pastures may have a large content. The drainage from highways, even in rural districts, may increase the bacterial content, and storm flows from village or urban districts carry great numbers of organisms into streams. Small lakes and artificial ponds in populous communities may have high bacterial contents, but large reservoirs and lakes are usually unfavorable habitats for many species and, therefore, the numbers are low. Many ground waters are practically free from bacteria.

Number of Bacteria Found in Excreta.—In his "Bacteriological Ex-

amination of Water Supplies," 1906, W. G. Savage gives data from Ford's researches which revealed large numbers of bacteria of about 50 species in the human alimentary canal. Dr. A. C. Houston found in 17 normal stools of healthy persons between 100 million and 1000 million bacteria per gram of feces, whether grown upon gelatine at 20°C. or upon agar at 37°C. Of these over 100 million were B. coli, between 1 and 10 million were spores of B. enteritidis sporogenes and about the same number virulent organisms, and at least 100,000 were streptococci. Animal excreta also teem with bacteria, some apparently identical with species found in human excreta.

Number of Bacteria Found in Sewage.—The total number of sewage bacteria capable of growing upon nutrient gelatine at 20°C. increases with the strength and the age of the sewage. The number may also be affected by industrial wastes discharged into the sewers. Wastes from packing houses and some tanneries contain materials stimulating the increase of bacteria, whereas pickling liquids from galvanizing works, being germicides, tend to reduce the number. Some industrial wastes offer serious difficulties in sewage treatment on account of their antiseptic nature. For example, some paper mill wastes are practically free from bacteria, and if organisms are introduced they will not multiply sufficiently in a reasonable length of time to be effective in causing biological changes, unless the wastes are diluted with river water or sewage, or treated chemically to overcome their germicidal nature.

The number of bacteria in municipal sewage at the point of discharge is much greater if the sewers are very long, laid with flat gradients, and contain organic deposits. Such conditions should not be overlooked for otherwise the interpretation of bacterial counts may be erroneous.

The sewage used at the Lawrence Experiment Station is practically free from industrial wastes and appears to be representative of domestic municipal sewage. Yearly averages of bacterial counts of weekly samples of it are given in the annual reports of the Board, as follows:

Year	1909	1910	1911	1912
Bacteria per cubic centimeter	1,136,400	1,507,300	2,033,100	2,109,600

The bacterial content of municipal sewage may be quite different from that from toilet rooms on account of dilution with water used for laundry, washing, and other household activities and with ground water leaking into the sewers. The difference was well illustrated at the Lawrence Experiment Station by the following bacterial counts of "representative samples of all the sewage from the toilet room at the station during one day each week" for two years:

Year	1909	1910
Bacteria per cubic centimeter	2,849,300	3,158,200

At Columbus, Ohio, in 1904 and 1905, George A. Johnson found that the total number of bacteria contained in the sewage (120 gal. per capita daily) ranged from 320,000 to 27,000,000, averaging 3,600,000 per cubic centimeter. (Report on Sewage Purification, page 53.)

At the Philadelphia Experiment Station in 1909 and 1910, sewage was received from about 1500 acres of built-up area which yielded "concentrated sewage and a large amount of trade wastes." In the report of the station's work, carried out under the direction of George S. Webster by George E. Datesman, with W. L. Stevenson in direct charge, the monthly averages of bacteria in the crude sewage, capable of growing upon nutrient gelatine at 20°C., range from 1,100,000 per cubic centimeter in January to 5,800,000 in August, the average from July 5 to April 30 being approximately 3,000,000. During March the average counts were:

Sunday	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday	Average
1,500,000	2,500,000	2,300,000	2,000,000	1,800,000	2,000,000	1,600,000	2,000,000

At the experiment station at Gloversville, N. Y., in 1908 and 1909, sewage (130 gal. per capita daily) from a 3-mile sewer serving about 20,000 persons and 25 small tanneries, contained about 1,600,000 bacteria per cubic centimeter capable of growing on gelatine at 20°C. (Report of Experiments by Eddy and Vrooman, page 126.) This rather low content may have been due to the antiseptic character of some tannery wastes.

English sewages contain as a rule more bacteria than are commonly found in American sewages, mainly because the former are much more concentrated on account of the use of smaller quantities of water.

In the fourth report of the Royal Commission on Sewage Disposal (page 210), the following statements are to be found:[1]

"This opportunity may be taken to point out that domestic sewage and mixed sewage, when the trade refuse is not of such amount or quality as to alter materially the biological qualities of the mixed liquid, commonly give bacteriological results as follows:

"Total number of bacteria, gelatine at 20°C. more than 10,000,000 but less than 100,000,000 bacteria per cubic centimeter.

"Total number of bacteria, agar at 37°C., more than 1,000,000, but less than 10,000,000 per cubic centimeter.

Hourly Variation in Bacteria in Sewage.—The bacterial counts in Table 6 illustrate the varying number of bacteria in sewage from hour to hour. In comparing such records, attention should be paid to the length of time required for the sewage to reach the sampling point, for the sample nominally represents the sewage entering the sewers that

[1] Quantity of sewage probably varied from 32 to 42 U. S. gal. per capita.

much earlier, although the lapse of time is sufficient in many cases to permit a substantial multiplication of bacteria.

TABLE 6.—NUMBER OF BACTERIA IN HOURLY SAMPLES OF SEWAGE FROM MARLBORO AND GARDNER, MASS.

(Report of the Mass. State Board of Health, 1894, pages 467 and 468)

Marlboro,[1] July 10, 1894				Gardner,[1] July 26, 1894			
Hour	Bacteria per cc.	Hour	Bacteria per cc.	Hour	Bacteria per cc.	Hour	Bacteria per cc.
11.00 A.M.	2,077,600	9.00 P.M.	1,440,000	12.00 M.	1,101,600	10.00 P.M.	795,600
12.00 M.	1,652,400	10.00 P.M.	1,396,000	1.00 P.M.	921,600	11.00 P.M.	918,000
1.00 P.M.	2,529,200	11.00 P.M.	1,219,600	2.00 P.M.	1,000,000	5.00 A.M.	238,000
2.00 P.M.	2,937,600	12.00 P.M.	979,200	3.00 P.M.	1,530,000	6.00 A.M.	204,000
3.00 P.M.	2,019,600	5.00 A.M.	960,000	4.00 P.M.	1,157,000	7.00 A.M.	856,000
4.00 P.M.	2,133,200	6.00 A.M.	633,400	5.00 P.M.	856,800	8.00 A.M.	2,142,000
5.00 P.M.	1,958,400	7.00 A.M.	633,600	6.00 P.M.	921,600	9.00 A.M.	1,346,400
6.00 P.M.	1,897,200	8.00 A.M.	570,000	7.00 P.M.	691,200	10.00 A.M.	1,409,600
7.00 P.M.	1,382,400	9.00 A.M.	675,200	8.00 P.M.	864,000	11.00 A.M.	1,897,200
8.00 P.M.	1,497,600	10.00 A.M.	979,200	9.00 P.M.	795,600	12.00 M.	1,409,000

Marlboro samples taken 4 miles below last house connection, Gardner samples taken from outfall sewer 1 mile from outskirts of town.

[1] Rate of sewage flow varied as follows:
Marlboro, 14 to 32, average 25 gal. per capita per day.
Gardner, 15 to 31, average 24 gal. per capita per day.

Collection of Samples.—It is highly desirable for the bacteriologist who is to examine the samples to collect them. If this is impracticable, the person collecting the samples should guard carefully against their contamination. They should be collected in bottles with glass stoppers, preferably of the flat mushroom type, holding for ordinary purposes about 4 oz. Prescott & Winslow ("Water Bacteriology," page 33) recommend wide-mouthed bottles. The bottles should be thoroughly cleansed in the laboratory with sulphuric acid and potassium bichromate, or alkaline permanganate of potash followed by sulphuric acid. Then they should be thoroughly rinsed with distilled water, dried by draining, and sterilized with dry heat for 1 hour at 160°C., or in an autoclave at 120°C. for 15 minutes. After sterilizing, the bottles should be wrapped in sterilized cloth or paper, or the stopper and neck should be wrapped with tinfoil, and the bottles placed in tin boxes for transportation.

When the sample is taken, the paper, cloth or tinfoil should be removed and the stopper withdrawn, taking care not to let the fingers touch anything which will later come into contact with the water collected. When collecting a sample of running water, the mouth of the bottle should be pointed upstream in order that contamination from the fingers may be carried away, instead of into the bottle. When collect-

ing samples from still water the same result may be attained by rapidly pushing the bottle through the water while it is being filled. When the sample is being taken from a tap or pump, the water should be allowed to run for a period of several minutes before the sample is collected. After the bottle has been filled, the stopper should be immediately replaced and the stopper or bottle rewrapped with the cloth, paper or foil originally protecting it.

After obtaining a sample of water, the number of bacteria in it may increase or decrease and it is, therefore, desirable to adhere to the recommendations of the Committee upon Standard Methods of Analyses, of the American Public Health Association, that the interval between sampling and examination should not exceed 12 hours in the case of relatively pure waters, 6 hours in the case of relatively impure waters and 1 hour in the case of sewage. If the sample must be transported a considerable distance before it is plated, the bottle should be packed in ice. This will prevent a rapid increase in the bacteria present, but it is to be remembered that long-continued exposure to low temperature is inimical to bacterial life.

Sewage a Nutrient Medium for Bacteria.—The organic and mineral matter in sewage usually affords suitable food for bacteria, which multiply in it with enormous rapidity at a favorable temperature, such as 20°C. This fact is useful in ascertaining whether a sewage containing industrial wastes is capable of undergoing the ordinary biological changes required for its successful treatment. A sample upon standing for some hours at 20°C. should show a great increase in the number of bacteria present, if the sewage is susceptible to such action.

The change in the number of bacteria in municipal sewage without antiseptic wastes is shown by the results of a test at the Lawrence Experiment Station reproduced in Table 7. Sewage taken at 10 A.M. was allowed to stand at 16° to 20°C. for 8 days. The rapid increase at first was doubtless due to an abundant and suitable supply of food. The decrease which followed was probably due in large measure to the diminution in the food supply and in the dissolved oxygen, which disappeared in less than 24 hours after the sample was taken. It is also possible that products of bacterial metabolism[1] inimical to bacterial development were generated in the sample.

Bacteria Capable of Growing at Body Temperature.—In some bacterial investigations, especially to detect contamination of water, it is desirable to encourage the growth of intestinal bacteria and discourage that of normal water bacteria, that the latter may not so outnumber the former as to prevent their detection. Many bacteria living within the

[1] The process by which living cells absorb their food and convert a portion of it into their structures and a portion into energy, a remainder being discarded as useless; the life processes of bacteria and other organisms.

intestines of warm-blooded animals grow better at body temperature, 37°C. (98.6°F.), than at ordinary room temperature, 20°C. (68°F.). Many bacteria thriving at 20°C. fail to multiply rapidly at 37°C. Accordingly it is customary in some laboratories to determine the bacteria count in contaminated water and in sewage at both temperatures.

TABLE 7.—CHANGES IN COMPOSITION OCCURRING IN A BOTTLE OF FRESH LAWRENCE STREET SEWAGE, UPON STANDING. ORIGINAL SAMPLE TAKEN MARCH 11, 1894

(Report of Massachusetts State Board of Health, 1894, page 461)

Elapsed time, hours	Nitrogen as free ammonia	Organic nitrogen: 14 Alb. Ammonia ÷ 17, Total	Organic nitrogen: 14 Alb. Ammonia ÷ 17, Soluble	Organic nitrogen: by Kjeldahl method, Total	Organic nitrogen: by Kjeldahl method, Soluble	Nitrogen as nitrates and nitrites	Oxygen consumed in 2 min.	Oxygen consumed in 5 min.	Dissolved oxygen, per cent.	Bacteria per cubic centimeter
Sample	1.85	0.80	0.58	4.03	3.14	0.35	8.50	13.20	57	1,190,000
2	2.06	0.82	0.58	4.03	2.89	0.31	8.10	14.20	60	1,085,000
4½	2.10	0.83	0.58	3.96	2.89	0.29	8.50	13.60	60	1,505,000
7½	2.35	0.83	0.56	3.77	2.78	0.25	8.50	13.30	30	1,530,000
21½	4.08	0.88	0.49	2.38	1.16	0.03	7.40	10.70	0	20,475,000
25½	4.12	0.92	0.49	1.98	1.16	0.02	6.90	9.80	0	23,100,000
30½	4.12	0.83	0.42	2.17	1.16	0.00	6.90	9.80	0	20,000,000
48	4.20	0.82	0.47	1.98	1.16	0.00	6.00	8.90	0	12,810,000
72	4.12	0.78	0.33	1.78	0.94	0.00	5.50	7.20	0	11,235,000
96	4.12	0.76	0.34	1.76	0.93	0.00	4.90	6.80	0	6,825,000
120	4.12	0.76	0.35	1.64	0.75	0.00	4.80	7.20	0	4,485,000
168	4.20	0.68	0.30	1.43	0.74	0.00	5.10	7.00	0	3,420,000
192	4.28	0.69	0.30	1.37	0.59	0.00	5.00	7.10	0	2,341,000

Results of three measurements of flow in this sewer, obtained through the courtesy of H. W. Clark, indicated flows varying from 130 to 210 gal. per capita and that perhaps 130 gal. per capita is about the normal rate.

In their "Twenty-one Years' Experiments at Lawrence" (1909) Clark and Gage state that there is an approximately constant ratio between the total counts at 20°C. and 40°C.[1] for all sewages and effluents from sewage filters under normal conditions, and therefore that the percentage removal of bacteria by filters computed from each count should be substantially the same and that the determinations at the two temperatures should act as a check upon one another. Hence, one of these determinations might be omitted where simply the efficiency of a filter or

[1] The Lawrence laboratory adopted 40°C. as the temperature for such tests many years ago, but the temperature of 37°C. now in common use more nearly corresponds to the temperature of the human body.

other process in removing bacteria is required. As the 37°C. counts show more nearly than those at 20°C. the numbers of bacteria of intestinal origin, which are of particular sanitary importance, the results obtained at the former temperature are more valuable in showing the hygienic efficiency of filtration systems, or the sanitary quality of the water. Another advantage of higher temperature is that colonies develop much more rapidly at 37°C. than at 20°C. so that the results are available much sooner. The period of incubation at 37°C. is commonly 24 hours or less, and at 20°C. from 48 to 72 hours.

Many bacteria normal to water and the soil, as well as some of those originating in the animal body, appear to bring about important changes in sewage. Therefore, it seems to be desirable to include both classes in many of the studies relating to the treatment of sewage, incubating gelatine plates at 20°C. and agar plates at 37°C.

Species of Bacteria Found in Sewage.—Comparatively little study has been devoted to the isolation, identification and life processes of the species of sewage bacteria. This lack of information is due, in part, to the difficulties of isolating and studying the bacteria, in part to the large number and variety present, and in part to the complex character of the sewage and the conditions affecting its bacterial content.

The most natural biological treatment of sewage appears to be through such control of conditions as will cause the vigorous predominant growth of species capable of bringing about desired changes. The conditions under which the desired bacterial action can be obtained are ascertained by studying the changes in the sewage rather than by examining its bacterial content at any stage of the treatment. In other words, the art of biological sewage treatment, today, is governed by more or less accurate knowledge of the work accomplished by masses of bacteria and the results to be obtained by producing different working conditions, rather than by a knowledge of bacteria themselves and of their individual characteristics.

Bacteria capable of living outside the human body, being at all times present in the excreta of man, are obviously ever present in fresh sewage. Certain types which are present in times of illness, as the cholera spirillum and bacillus typhosus, are obviously present in fresh sewage to which patients suffering from these maladies contribute. These patients ordinarily bear so small a relation to the tributary population that the proportion of such organisms to the total number of bacteria in municipal sewage is extremely small, which, together with the difficulties of isolation and identification, is sufficient reason for the usual failure to find such organisms in sewage.

Dr. Samuel Rideal, in "Sewage and the Bacterial Purification of Sewage" (1906, page 72) gave a list, Table 8, of 65 different bacteria found in sewage.

TABLE 8.—BACTERIA OCCURRING IN SEWAGE

(Rideal, "Sewage and the Bacterial Purification of Sewage," 1906)

Note.—L, Liquefying gelatine; NL, not liquefying; SL, slightly liquefying.

OBLIGATORY ANAEROBES

Bacillus amylobacter, L (Clostridium butyricum).

B. enteritidis sporogenes, L; cadaveris sporogenes, L; butyricus, L. Give much gas.

Spirillum rugula, L. Very active; gives rise to fecal odor.

S. amyliferum. Acts as a vigorous ferment.

FACULTATIVE ANAEROBES OR AEROBES

B. fluorescens liquefaciens, L, and non-liquefaciens, NL; megaterium, L; magnus, spinosus, liquefaciens, L; mesentericus, L. Several varieties in London sewage produce H_2S.

B. vermicularis, L; liquidus, L; ramosus, L; mycoides, L; fuscus, NL; nubilus, L; cloacæ, L; ubiquitus, NL; reticularis, SL; cereus, L; circulans, L; hyalinus, L. All reducing nitrates to nitrites and NH_3.

B. aquatilis, SL (grows luxuriantly in ammonia solutions); brunneus, NL; helvolus, L; and superficialis, SL. Not reducing nitrates.

B. saprogenes, I, II, III; pyogenes and coprogenes fetidus.

B. putrificus coli, NL; fluorescens putridus, L. Decompose albuminous substances, liberating NH_3.

B. coli communis, NL; acidi lactici, NL; lactis aerogenes, NL. All producing gas.

B. subtilis, L. Forms highly resistant spores; rapidly consumes oxygen.

B. sulphureum, L. Liquefies casein; produces H_2S.

B. lactis cyanogenus, NL; erythrosporus, NL; rubescens, NL; pyocyaneus, L (a culture from London sewage proved extremely virulent).

Several varieties of thermophilic bacilli, capable of luxuriant growth at temperatures above 50°C., and producing spores.

Micrococcus (and Bacillus) ureæ, NL; ureæ liquefaciens, L. Converting urea into ammonium carbonate.

M. tetragenus mobilis ventriculi, NL. reduced nitrates.

M. casei, NL; albicans amplus, L; fervidosus, NL.

Streptococcus mirabilis, NL; vermiformis, L; coli gracilis, L; liquefaciens coli, L.

Spirillum plicatile, serpens, undula, tenue, and volutans.

Sarcina alba and lutea, SL.

Proteus vulgaris, L; produces NH_3 from nitrogenous organic matter.

Proteus mirabilis, L; zenkeri, L; sulphureus L. (produced NH_3 and mercaptan).

Beggiatoa alba. Secretes granules of sulphur, formed, according to Winogradsky, by oxidation of H_2S, and finally turned into H_2SO_4 by the plant.

Three groups of sewage bacteria will be considered and the important work done by, and tests for, them discussed.

I.—Bacteria capable of decomposing and oxidizing the organic matter of sewage. In this class would be included the nitrifying organisms.

II.—Bacteria which serve as test organisms by means of which knowledge may be acquired of the probability of past, present or future contamination of relatively clean water, with pathogenic bacteria. Among such organisms B. coli communis is probably the best known.

III.—Pathogenic bacteria, like the cholera spirillum and B. typhosus.

BACTERIAL DECOMPOSITION AND OXIDATION OF SEWAGE

The growth and activities of bacteria are dependent upon the consumption of food, as with every other form of life. It is to the metabolism of minute organisms, including plankton, that the transformation of obnoxious and offensive sewage to unobjectionable water is due.

Theory of Bacterial Action.—The bacterium is a minute cell composed of the enclosing wall and the protoplasm inside. The protoplasm can extract and assimilate nourishment from the food reaching it, portions forming new cell wall and more protoplasm, resulting in growth, and other portions furnishing vital energy.

The cell wall permits transfusion of soluble substances but keeps out solid particles. Since much bacterial food is in solid form, the organism must dissolve this food outside the cell, which is done by substances called enzymes, secreted by the cells. The theory of enzyme action is explained by Rahn as follows:

"It has been stated before that many micro-organisms feed upon cellulose, starch, fat, gelatin, keratin and other insoluble compounds. It has also been previously stated that micro-organisms, with the exception of some protozoa, depend upon soluble food, since they have no means of incorporating insoluble compounds into their protoplasm. The protoplasm, however, must be considered the center of metabolism, and the digestion of food and the formation of energy must take place in the protoplasm if the cell is to profit by it. Since the food cannot diffuse into the cell, and the protoplasm does not diffuse out, the food must be dissolved. This is accomplished by the cell itself, which secretes certain agents having peculiar qualities. These agents, the so-called enzymes,[1] act upon the insoluble foods, changing them into soluble compounds which then can diffuse into the cell, where they are digested or fermented. The final digestion or fermentation of the food

[1] "The organic ferments which have been isolated are amorphous, albumin-like substances; it is usually impossible to remove them from organic tissues with which they are associated. During reaction they must be either in the form of a gel on the surface of the tissue, or in the form of a sol (probably emulsoid or hydrophile). In either case the action is obviously heterogeneous, and absorption, surface concentration, and diffusion must therefore take an important part in the changes. If the chemical change is a rapid one, the Nernst-Brunner principle must also apply, and if the enzyme is a sol, Brownian movement will come into play, as will also changes in the specific surface under the influence of reagents, especially of acids, alkalies and salts." ("Chemistry of Colloids," by W. W. Taylor, page 298.)

must take place within the cell. Energy production outside the cell serves the same purpose as a stove outside the house. The dissolution of insoluble compounds by cell secretions must be considered a preparatory process which has no direct relation to intracellular food digestion or fermentation." ("Microbiology," edited by Marshall, page 130.)

While some enzymes have been known for many years, even before the organisms secreting them were known, there is still no knowledge as to their chemical composition. They possess the peculiar property of not being destroyed by the processes in which they take part. Like the organisms which secrete them, they are sensitive to heat and poisons, becoming inactivate on exposure to temperatures above 50° to 80°C. and to disinfectants like formaldehyde and mercuric chloride.

The digestive processes of micro-organisms are described by Rahn as follows:

"The final decomposition, the process which yields the energy for cell life, must take place within the cell. The investigations of recent years have demonstrated that these processes also are caused by enzymes. It has been proved beyond doubt that in the alcoholic, lactic, acetic and urea fermentations, the fermentation process may continue after the death of the fermenting cells. In the case of alcoholic fermentation, the fermenting agent has been separated from the lacerated cells and has been filtered through porcelain filters without losing its ability to act. This proves the enzyme nature of the fermenting agent which, after once being formed, remains and acts independent of the cell. These enzymes are called 'zymases.' They remain within the cell as long as it is alive. They are much more sensitive to injurious influences than the above-mentioned food-preparing enzymes. Much skill and patience was required to demonstrate their independence of the living cell. After these enzymes were found in micro-organisms, similar enzymes were discovered in the cells of higher plants and animals. Many of the bio-chemical changes taking place in the final dissociation of food within the cell are now known to be the result of enzymic action; heretofore these reactions were believed to be a part of the life-processes, inseparable from the living cell. Even some of the oxidations and many reducing processes have been recognized as caused by enzymes, and it is quite possible that the whole process of intracellular food decomposition is accomplished entirely by means of enzymes." ("Microbiology," edited by Marshall, page 134.)

In the absence of knowledge of the chemical nature and action of enzymes, they are classified roughly by Rahn according to their action as hydrolyzing, zymatic, oxidizing and reducing. His discussion of the subject in Marshall's "Microbiology" may be outlined as follows:

Enzymes which decompose carbohydrates[1] break up complex mole-

[1] Carbohydrates are complex organic compounds, made up only of carbon, hydrogen and oxygen atoms, containing six, or a multiple of six, carbon atoms, and hydrogen and oxygen atoms in the same ratio as in water, that is, two atoms of hydrogen to every atom of oxygen.

cules into simpler ones by adding water, whence the name "hydrolysis." An illustration is the hydrolysis of cellulose to soluble sugar, expressed by the following formula:

$$\underset{\text{Cellulose}}{(C_6H_{10}O_5)_n} + \underset{\text{Water}}{n(H_2O)} = \underset{\text{Sugar}}{n(C_6H_{12}O_6)}$$

Enzymes decompose fats in the same manner. There appear to be but few organisms which decompose fats and they convert them into fatty acids and glycerin by the addition of three molecules of water. For example:

$$\underset{\text{Tristearin}}{C_3H_5(C_{18}H_{35}O_2)_3} + 3H_2O = \underset{\text{Stearic acid}}{3(C_{18}H_{35}O_2H_3)} + \underset{\text{Glycerin}}{C_3H_5(OH)_3}$$

Nitrogenous organic substances, proteins, are decomposed similarly by the proteolytic enzymes, a characteristic of which is their ability to liquefy gelatin.

There are also bacterial enzymes which will coagulate dissolved substances, as the bacteria found in milk which form rennet and precipitate the casein.

Of the zymases, urease, which decomposes urea with formation of ammonium carbonate $(NH_3)_2CO_3$, was the first to be recognized.

Certain enzymes, oxidases, are produced which apparently bring about their changes by oxidation. For example, vinegar-oxidase converts alcohol to acetic acid according to the following reactions ("Microbiology," page 448):

$$C_2H_6O + O = C_2H_4O + H_2O$$
$$\text{Ethyl alcohol} + \text{Oxygen} = \text{Acetic aldehyde} + \text{Water}$$

$$C_2H_4O + O = C_2H_4O_2$$
$$\text{Acetic aldehyde} + \text{Oxygen} = \text{Acetic acid}$$

Enzymes having the reverse action of oxidases and called reductases, are common. Their changes are wrought through the abstraction of oxygen from the complex molecule. For example, nitrates are sometimes reduced progressively to nitrites, ammonia and nitrogen and the action is carried out by bacteria through the instrumentality of reductases secreted by them. Rahn states:

"These processes will take place after the cell is killed by a disinfectant or is ground to pieces. This can be readily demonstrated by lacerating the cells with quartz sand. They will then reduce nitrates to nitrites, sulphur to hydrogen sulphide. The decolorization of litmus, methylene blue, indigo, and other organic dyes is due in microbial cultures to enzymes which are almost exclusively endo-enzymes.

"The enzymes are mostly influenced by their own products, and when a

certain yeast ceases to ferment sugar at the concentration of 8.5 per cent. of alcohol, this means that the alcoholase of this yeast cannot tolerate more than 8.5 per cent. of alcohol. The inability of the cell to regulate enzymic action may account for the fact that often a culture produces an amount of fermentation products sufficient to kill all cells. This is observed in the lactic, acetic and alcoholic fermentations, and perhaps occurs in many others. Most cells produce more than one enzyme. Micro-organisms feeding upon various foods must form various enzymes. Frequently several enzymes are necessary for the decomposition of one compound." ("Microbiology," edited by Marshall, page 143.)

Rotation of the Elements in Nature.—Students of the problems of sewage disposal must remember always that matter is indestructible; that whatever changes may be wrought in its form and characteristics, 100 lb. of original substance will always remain 100 lb. When living organic matter dies, decay due to micro-organisms causes a rearrangement of the chemical elements in it, into other substances. These may be changed through many transformations, but finally these same elements reappear in living organisms.

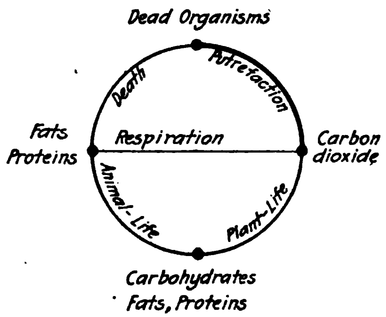


FIG. 6.—Carbon cycle (after Rahn). (From "Microbiology," edited by Marshall, page 125.)

Rahn illustrates this rotation of the elements in the carbon cycle by Fig. 6. At the death of the organisms, the first stage is that of decay, when putrefaction may perform an important function. This stage ends with the oxidation of the carbon into carbon dioxide gas or carbonates. In this form it is food for green plant life and is converted into such complex organic substances as carbohydrates, fats and proteins. The mature plants finally die and the elements of which they are constituted may at once recommence the cycle, or the plant may serve as food for animal life, by which it is converted into such animal tissues as fats and proteins.

Breaking Down of Complex Substances.—The bacterial decomposition of sewage is often carried out by several species of organisms working, so to speak, in relays. Each utilizes in turn some of the decomposition products of the metabolism of the preceding organisms. If it were possible to control these processes, sewage might be passed from tank to tank or filter to filter, in each of which only one process would take place.

The multiplication of bacteria in sewage is attended by the formation of ammonia, carbon dioxide, and sometimes hydrogen, nitrogen

and methane or marsh gas (CH_4). Some organisms produce hydrogen sulphide (H_2S), as described more fully later. There is some evidence that the more complex and stable matter, which yields its nitrogen on analysis only in response to the vigorous Kjeldahl test, is first changed into less complex nitrogenous substances responding to the test for albuminoid ammonia, and that these latter substances are decomposed to form free ammonia and its salts. These biological reactions are very complex and little is known of them.

Sulphur Bacteria.—Another process of evolution of much interest and importance in problems relating to the disposal of sewage is the sulphur cycle.

Sulphur in sewage may be derived from any one or all of the following sources: (1) Vegetable or animal organic matter. Albumin is such a substance, that of the egg being typical. Everyone is familiar with the characteristic odor of the rotten egg, due to hydrogen sulphide (H_2S). (2) Sulphates of calcium and magnesium, often dissolved in ground water leaking into sewers and sometimes in water supplies. (3) Sulphates and sulphides of the alkalies, of the alkaline earths and of metals present in some industrial wastes.

Sulphur is always present in sewage ready to enter into any biological process, although varying greatly in quantity according to its source. Bacterial development in sewage is so rapid that the dissolved oxygen is soon exhausted, and the organisms appear to appropriate the oxygen combined with the mineral and organic matter, hydrogen sulphide being split off. If sulphates are present they will yield their oxygen, and the resulting sulphides may be decomposed with the formation of hydrogen sulphide. Such reduction[1] may prove a source of seriously offensive odors where sewage is allowed to undergo anaerobic decomposition.

An illustration of the reduction of sulphates was observed by the authors at Worcester, Mass., where great quantities of ferrous sulphate, from the dipping of iron wire into sulphuric acid, are discharged into the sewers. This sewage, while at times rather strongly acid, will support bacterial life. When rapid growth is encouraged, the bacteria quickly exhaust not only any dissolved oxygen present but also that in combination with other elements. In this way the ferrous sulphate ($FeSO_4$) is deprived of its oxygen and ferrous sulphide (FeS) remains. The affinity of iron for sulphur, however, is so great that under ordinary conditions the sulphide of iron remains intact and does not split up with the formation of hydrogen sulphide. This fact may explain the failure of some sewages to evolve hydrogen sulphide, even when anaerobic bacterial action is vigorous.

Hydrogen sulphide may result from the action of many kinds of

[1] Reduction in the sense here used means the transformation of a substance containing oxygen to one of less or no oxygen content.

bacteria. On the other hand Jordan points out ("General Bacteriology," fourth edition, page 97) that the reduction of sulphates is a quality not very widely shared and one not possessed by such bacteria as B. coli, although this organism reduces nitrates vigorously. Beijerinck has isolated an organism which he calls spirillum desulfuricans and which he regards as the peculiar organism of sulphur reduction. Phelps mentions B. sulphureus and states that a non-liquefying anaerobic bacillus was isolated from Boston sewage by G. R. Spaulding, which reduced sulphates strongly and he considers this action common to many species. ("Microbiology," edited by Marshall, page 218.)

After the organic sulphur-bearing compounds become mineralized, with the formation of hydrogen sulphide, which does not appear suitable for the nutrition of the higher plants, another transformation is brought about by the so-called sulphur bacteria, capable of oxidizing the hydrogen sulphide. Such organisms were

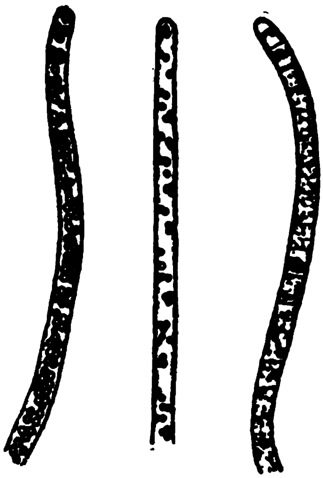


FIG. 7.—Beggiatoa alba (after Winogradsky, from Schmidt and Weiss). (From "Microbiology," edited by Marshall, page 60.)

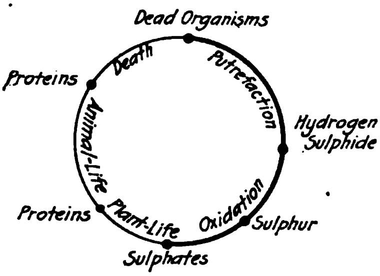


FIG. 8.—Sulphur cycle (after Rahn). (From "Microbiology," edited by Marshall, page 127).

long ago found in spring waters containing hydrogen sulphide and they may be found wherever there is an abundant supply of hydrogen sulphide and of oxygen, upon both of which their growth and activity seem to depend. Among these bacteria may be mentioned the Beggiatoa, the Thiothrix and the Rhodobacteriaceæ of Winogradsky. The Beggiatoa, Fig. 7, are highly motile cylindrical filaments; the Thiothrix do not possess motility; while the Rhodobacteriaceæ-form red or purple pigments. All appear to consume hydrogen sulphide as a food and do not seem to require organic matter, for they convert hydrogen sulphide into sulphuric acid, which is promptly neutralized by any alkaline substances at hand. Winogradsky found that the Beggiatoa consume daily from two to four times their weight of hydro-

gen sulphide. If removed from its presence the sulphur stored in the protoplasm quickly disappears and they apparently die of starvation.

In this way the sulphur cycle is completed by the oxidation of the hydrogen sulphide to sulphates, which serve as plant food. These changes are illustrated by Fig. 8.

Production of Hydrogen Sulphide.—In 1903, Clark investigated at the Lawrence Experiment Station the effect of sulphates in sewage upon the production of the offensive odor of hydrogen sulphide. Sewage to which magnesium sulphate and other salts had been added was passed slowly through a tank. His report states:

"From the beginning of operation of the tank there was a very voluminous formation of gas. The effluent had a much darker color than that from other septic tanks in operation at the station, and also had the very strong and disagreeable odor of hydrogen sulphide, the formation of this hydrogen sulphide undoubtedly being due to the decomposition of the large amount of sulphate introduced into the sewage. . . . . The operation of the tank, however, has shown more or less clearly the reason that septic tanks in different localities produce different odors, this experiment indicating that it is perhaps not so much due to different bacterial actions or different organic matter, especially when the sewage is domestic sewage, but rather to the presence of sulphates in the water or sewage. The sludge accumulating in this tank was of an unusually offensive quality." (Report Mass. State Board of Health, 1903, page 251.)

In an investigation of the sewage of Melbourne, Australia, Dr. Thomas Cherry found that hydrogen sulphide was formed from the sulphates in the sewage, by organisms belonging chiefly to the non-liquefying species, which produced hydrogen sulphide only when deprived of atmospheric oxygen. Of these organisms the most numerous and important was B. coli communis,[1] which constituted 5 to 10 per cent. of the organisms present.

As a result of experiments with samples varying up to about 400 gal., Cherry reported that the quantity of hydrogen sulphide formed was closely related to the quantity of suspended matter in the sewage; that its production might be prevented by passing a small stream of air continuously through the sewage; and that the addition of artificial cultures of rapidly liquefying bacteria to the raw sewage brought about the liquefaction of all solid particles without the production of hydrogen sulphide.

To prevent the formation of the hydrogen sulphide, he suggested introducing into the sewage enough liquefying bacteria to prevent the bacteria capable of forming hydrogen sulphide from gaining the ascendency. (Report of Engineer-in-Chief, Melbourne and Metropolitan Board of Works, 1901–1903, page 133.)

[1] This statement appears to contradict that of Jordan, page 99, and leads to the impression that further investigation of this subject is desirable.

Progress of Decomposition with Diminution of Oxygen.—When sewage first reaches the public sewers it is "fresh" and contains free oxygen so that aerobic bacteria thrive. The bacterial activity rapidly depletes the oxygen, which may become exhausted, as sewage, unless vigorously agitated, absorbs oxygen from the atmosphere very slowly. This depletion of oxygen is particularly marked if the sewers are of such length that several hours elapse before the major part of the sewage reaches the outlet. Such sewage is classed as "stale," and while aerobic bacteria are still present they may be fast losing their vitality. At the same time the facultative anaerobes begin to increase in number and activity. Soon the last traces of oxygen disappear, the sewage becomes very dark in color or even black, and hydrogen sulphide or other sulphides are formed. Such a sewage is often called "septic." Obviously there is no sharp dividing line between fresh, stale and septic sewage.

When sewage stands for a long time or flows very slowly through tanks, the anaerobic organisms gain the ascendency, and ammonia, marsh gas, carbon dioxide, hydrogen, nitrogen and other gases may be formed as the complex organic compounds are broken down into simpler ones. This is a reduction or septic process.

Liquefaction.—Liquefaction,[1] the change of insoluble substances into soluble ones, has been described as a result of bacterial activity through enzymes. While comparatively little is known concerning the manner in which it may be brought about on a large scale and under such control that full advantage may be taken of the process, it takes place under many conditions and is believed to be an important part of various processes of sewage treatment.

Certain species of bacteria appear able to liquefy in much greater measure than others. Rideal gives the number of liquefying germs in the crude sewage at Chorley, England, as between 20,000 and 1,000,000 per cubic centimeter out of a total of approximately 4,000,000 bacteria.

Oxidation Processes.—While oxidation goes on to some extent during anaerobic decomposition, it is usual to consider aerobic conditions essential to oxidation which approaches completeness. As anaerobic action, generally considered synonymous with putrefaction, frequently, if not always, produces offensive odors, it should usually be avoided. It has sometimes been held that putrefaction was conveniently, if not necessarily, precedent to oxidation, but this theory seems to have lost much support.

The process of oxidation of free ammonia first to nitrites and then from nitrites to nitrates by separate and distinct organisms, is called

[1] Generally the term liquefaction is so used as to include gasification, or the resolution of organic solids into gases.

"nitrification." That nitrification is due to bacterial action is apparent from the fact that it is checked or stopped entirely by low temperatures, by heating to the point of sterilization, and by certain chemicals inimical to the life of bacteria. Furthermore, temperatures favorable to bacterial development cause nitrification to proceed rapidly.

Oxidation of organic matter is effected by many aerobes, including those producing nitrites and nitrates. In sewage farming and intermittent filtration through beds of sand and other fine material, the production of high nitrates is an evidence of practically complete oxidation. Where sewage is applied to beds of very coarse materials and conditions do not closely simulate those of the natural soil, nitrates are not always found in such large quantities and consequently organisms producing them are not always regarded as so essential or so important as formerly.

Nitrosomonas or Nitrite-formers.—A bacterium called Nitrosomonas, capable of oxidizing ammonia to nitrites, was discovered by Winogradsky in 1888. According to Rideal it appears as circular corpuscles less than 1 μ in diameter, and sometimes as oval cocci ("Sewage," page 76). Similar nitrite-formers, each differing somewhat from the rest, have been isolated by different observers, but according to Jordan, no two forms have been found in one locality.

Nitrosomonas is peculiar in that, like the sulphur oxidizing bacteria, it will thrive on inorganic media and is unfavorably affected by organic matter. Its power to convert ammonia into nitrite appears to be dependent upon the presence of carbon dioxide, which it is said to assimilate in proportion to the quantity of nitrogen oxidized.

Nitromonas or Nitrate-formers.—Like the nitrite-formers the organisms producing nitrates are capable of growing on inorganic media. They can obtain their carbon from carbon dioxide or loosely bound carbonates, and while not so sensitive to organic matter as are the nitrosomonas, its presence interferes with their development although not so much with their power of oxidizing nitrites. Ammonia has a harmful effect upon them, a remarkably small quantity restraining their development. The nitrate-formers appear to act simply upon the nitrite present, adding thereto another atom of oxygen, converting it into nitrate, as for example sodium nitrate ($NaNo_3$). Jordan states:

"The fact that the transfer of a small quantity of material from a solution that had undergone nitrification sometimes induced complete nitrification in a fresh solution, sometimes carried the process only to the formation of nitrites, and sometimes failed altogether, was at first explained by assuming that the nitrifying organism had become weakened, but the discovery that the production of nitrites and that of nitrates were independent processes, due to the activity of distinct organisms, finally afforded the true explanation. Nitrites cannot be formed unless the nitrite-forming

organism is present; if the nitrite-former alone is present, the process stops midway; if the nitrate-former alone, the first step cannot be taken." ("General Bacteriology," page 564.)

In view of this statement and of the complex nature of sewage, with an ever present content of free ammonia and a substantial quantity of organic matter, it is at first difficult to understand how these sensitive nitrifying organisms can thrive and carry on their work of oxidation. The explanation is given by Jordan:

"The fact that under varied natural conditions the process of complete nitrification goes on steadily and quite rapidly is not in reality out of accord with the singular physiologic qualities and limitations of the nitrifying organisms. Although neither nitrite nor nitrate-forming organisms are able to generate ammonia from organic substances, they are constantly in association with myriads of bacteria in the soil and in water which do produce ammonia in abundance. Given ammonia as a starting-point, nitrite formation can occur, provided too much organic matter be not present, while after nitrite makes its appearance, the nitrate-former can pursue its activity. Ammonia seems to be injurious, especially to the development of the nitrate-former, less so to its oxidizing activity; however, both phases of the nitrifying process can go on simultaneously, provided living cells of the nitrate-forming organism are abundant at the time nitrite is produced. There is, hence, no real contradiction between the phenomena of nitrification observed under natural conditions and the characteristics of the nitrifying organisms exhibited in pure culture." ("General Bacteriology," page 567.)

A somewhat different view is presented by Clark and Gage in discussing the very slow nitrification of ammonia by pure cultures. They believe that nitrification, especially the rapid nitrification which occurs in some sewage filters, generally results from the associative action of a number of species of bacteria, any one of which alone would produce results but slowly.

Denitrification.—Under certain conditions it appears that a process of deoxidation or reduction may become established in some types of sewage filters. By this process the nitrates and nitrites appear to lose oxygen, and ammonia or nitrogen, or both, may be liberated. Such a change may be due to purely chemical reactions between products of bacterial activity and the highly oxygenated nitrogen compounds, or it may be a direct physiological process of some species of bacteria. In any event, a number of species of bacteria are capable of producing such changes by one method or another. Clark and Gage found:

"Some cultures were able to reduce the nitrates to nitrites, ammonia and elementary nitrogen continuously from the start, while with other cultures the reduction to these various bodies occurred consecutively, and with still others one or another of the reduction products was not formed

during the period over which the examination of the cultures extended." (Rept. Mass. St. Bd. Health, 1908, page 535.)

Jordan says that the "true" denitrifying bacteria, those able to reduce nitrates with the formation of free nitrogen as an end-product, are relatively few in number, but include such well-known species as B. coli, B. typhosus, B. fluorescens, and B. pyocyaneus. ("General Bacteriology," page 568.) They grow under both aerobic and anaerobic conditions, provided nitrates or nitrites are present.

The foregoing discussion may be summarized with the aid of Fig. 9, illustrating the nitrogen cycle, similar in many respects to the carbon and sulphur cycles already described. The dead organism, through the process of decay, sometimes accompanied by putrefaction, is decomposed and the nitrogen appears first as free ammonia. Later oxidation proceeds and the ammonia is nitrified, first to nitrites and then to nitrates. In the latter form it is good food for plant life and is accordingly converted into living complex nitrogenous organic matter or proteins. Such plants may die and decay or they may be consumed by animals, a part being converted into animal tissue which will later die and decay. Another part is converted into waste substances of which urea is representative. Some proteins may thus take a short cut through urea, which is easily oxidized, directly to ammonia, as illustrated by the chord of the circle. The processes of denitrification and fixation of nitrogen represented by the reentrant angle are commented upon by Rahn, as follows:

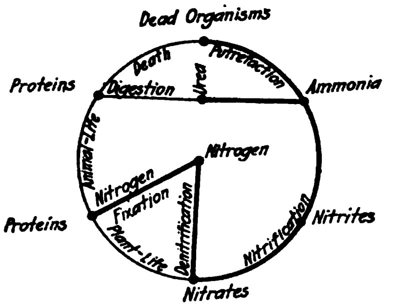


FIG. 9.—Nitrogen cycle (after Rahn). (From "Microbiology," edited by Marshall, page 126.)

"There is, however, one discrepancy in this cycle. It has been mentioned already that some organisms are able to reduce nitrates to nitrogen gas. This is one of the 'leaks' in the rotation of elements which would be disastrous to organic life on earth if there were no means to compensate for the loss of nitrogen in circulation. Imagine what would happen if there were no such compensation. Part of the nitrate in the soil is destroyed, the nitrogen gas escapes into the air and is as indifferent as the nitrogen of the atmosphere, lost to organic life forever. More nitrate would be produced from decaying organic matter and would be destroyed. After a certain time this continuous loss of nitrogen would become quite noticeable in the growth of plants; there would be a scarcity of nitrogen in the soil, since part of it is lost continuously. Finally, the plants would cease to grow because

the nitrogen in the soil would be exhausted. The compensation for this destruction of available nitrogen is found in the nitrogen-fixing bacteria, which . . . . have the power to use the atmospheric nitrogen for the formation of their own protoplasm. Thus, organic nitrogen is produced from nitrogen gas and the constancy of organic life is guaranteed." ("Microbiology," edited by Marshall, page 126.)

Offensive Odors.—Fresh sewage possesses a somewhat disagreeable odor, probably due mainly to compounds contained in the matter discharged into the sewage. It is not as objectionable as the odor from the anaerobic decomposition of sewage, and it is generally considered that the danger of producing offensive odors is much less where sewage can be treated while fresh.

The anaerobic decomposition of sewage, as already explained, may result in the production of hydrogen sulphide. However, other gases and certain volatile substances, such as indol, skatol, cadaverin and mercaptan, may cause odors even more offensive than that of hydrogen sulphide. There is little information at present as to their presence in sewage or the manner in which they may be produced by its decomposition. If sewage can be supplied with dissolved oxygen, its decomposition will not result in the production of the offensive odors so evident when undergoing anaerobic changes. Therefore, an effort is now generally made to bring about the necessary changes in sewage under aerobic conditions.

DETECTION OF CONTAMINATION BY TEST BACTERIA

In many instances the source of diseases like cholera and typhoid has been clearly traced to the drinking of water contaminated by the discharges from patients suffering with these diseases. In other cases, eating shellfish grown or fattened in similarly contaminated water has been found to be the cause. It is important, therefore, that methods should be available to detect such contamination in water.

Analyses are obviously needless when one can see sewers discharging into the water, but it may be desirable to know the degree of contamination. If the amount of sewage is large compared with the amount of water into which it is discharged, and the problems resulting are those of pollution rather than of contamination, chemical analyses will generally afford more useful information than bacterial examinations. When the investigations relate to water supply and the contamination of shellfish, the investigator must resort to bacterial analyses, which are capable of disclosing the presence of much smaller proportions of sewage than can be detected by chemical analyses. The contamination is traced by species of bacteria originating in the intestinal tract.

Typical Intestinal Bacteria.—It is important to select as the test organism a species present at all times in excreta from both well and

sick persons. Such organisms exist in vastly greater numbers than the germs of disease in sewage and contaminated water, because the persons suffering from a water-borne disease, from whom the pathogenic

Fig. 10.—Bacillus coli; twenty-four-hour agar culture; × 650 (Heim). (From Jordan's "General Bacteriology," page 264.)

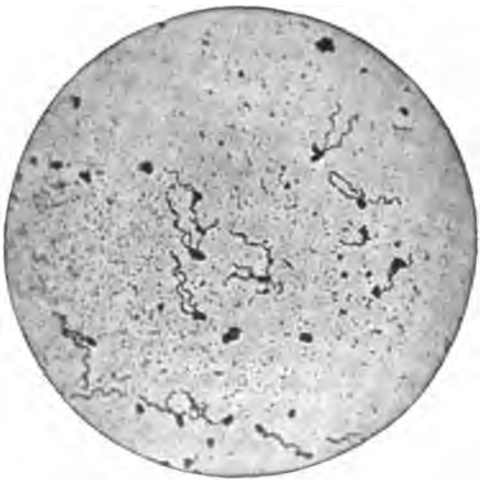


Fig. 11.—Bacillus coli with flagella stained by van Ermengem's method; × 1000 (Williams). (From Jordan's "General Bacteriology," page 264.)

organisms come, are few in comparison with the total contributing population.

Bacteriologists recognize three important groups of sewage bacteria: B. coli, B. enteritidis sporogenes and sewage streptococci. Clark and Gage state:

"Of these B. coli has formed a basis for the greatest amount of work. Nevertheless, the other two have considerable standing among various observers in different parts of the world. B. coli is supposed to show present or past pollution, usually recent pollution. The streptococci type, it is stated, should show only very recent pollution by sewage; evidence as to this type, however, has not been forthcoming. The bacillus sporogenes being a spore-forming organism may show present pollution and it may also show pollution which has occurred at some previous time and from which the danger has not entirely ceased." (Rept. Mass. St. Bd. Health, 1902, page 264.)

Jordan states, however, that the weight of available evidence is against the view that streptococcus shows recent and hence specially objectionable pollution, and that the longevity of streptococci in water is sometimes greater and sometimes less than that of the colon bacillus. ("General Bacteriology," page 596.)

B. coli is universally found in human urine and feces. According to Jordan it is 2 to 4 μ long and 0.4 to 0.7 μ broad and coccus-like forms occur (Fig. 10). The most typical B. coli are motile and their flagella[1] are shown by suitable stains (Fig. 11).

Clark and Gage found at the Lawrence Experiment Station (report State Bd. Health, 1902, page 265) that positive tests for B. coli and sewage streptococcus were obtained in the following percentages of samples:

Dilution, 1 to....................	10,000	50,000	150,000	1,000,000
B. coli, positive tests, per cent......	95	78	40	6
Streptococcus, positive tests, per cent.	67	6	0	0

They found that B. sporogenes can usually be detected in dilutions of 1:500 to 1: 1000, but very rarely detected in greater dilutions. B. coli apparently affords a more sensitive test than either the sewage streptococcus or B. sporogenes, and has become the commonly accepted test organism upon which to base judgment as to the presence and degree of sewage contamination. This test is also employed as a measure of efficiency of processes of water purification and sewage treatment where the removal of pathogenic organisms is an important requirement.

Variety of Organisms Constituting B. coli Group.—The organisms in the B. coli group are classified and described in "Standard Methods of Water Analysis" of the American Public Health Association (1912, page 80). This gives 17 varieties, 4 of which, however, have not as yet been isolated and described. The remaining 17 varieties are grouped in 4 species, B. communior (Durham), B. communis (Escherich), B. aerogenes (Escherich), and B. acidi lactici (Hueppe). It is stated in the

[1] Flagella are long, fragile filamentous appendages which drive the bacterium through the water by virtue of their power of contractility ("General Bacteriology," Jordan, page 63).

same place that the so-called typical B. coli does not exist as such, but that the entire group is typical of the presence of fecal matter when water or sewage examinations are to be considered.

Tests for B. coli.—B. coli has two marked general characteristics utilized in testing for it; it produces acid and gas which, under proper conditions, are highly significant.

To test for B. coli the sample of water is plated, after mixing with nutrient agar containing 1 per cent. sugar (lactose) and a small quantity of litmus, which colors the agar blue. After incubation at 37°C. for 18 to 24 hours, a plate containing B. coli will disclose colonies red in color, due to the action upon the litmus of the acid produced by the B. coli. Not all the red colonies, however, are necessarily bacteria of the B. coli group, because some other species also produce acid.

The test is carried further by transplanting minute portions of the red colonies into U tubes closed at one end and filled with nutrient broth containing lactose and ox bile. These tubes are incubated at 37°C. for 48 hours. If the colonies transplanted were of the colon group, gas will accumulate in the closed end of the tube, the quantity averaging generally about one-half the capacity of the closed arm of the tube. Of this gas about one-third will be carbon dioxide, which may be determined by absorption with potash solution. As attenuated B. coli does not represent recent contamination and as lactose bile medium is slightly inhibitive to B. coli, especially in attenuated form, any positive tests with this medium indicate recent or fresh contamination. ("Standard Methods of Water Analysis," page 87.)

The fact that all B. coli produce gas when cultivated in sugar broth and red colonies when cultivated upon lactose-litmus-agar plates, together with the fact that very few other bacteria do produce the red colonies and the same proportion of gas and of carbon dioxide, have led to the practice of considering these as presumptive tests, the presumption being that bacteria giving positive results with one or both of these tests are B. coli.

The presumptive tests, however, are not conclusive and in many cases the investigation should be carried further, to establish the following characteristics:

(1) B. coli does not liquefy gelatine. This is demonstrated by making a stab culture, that is, by thrusting an inoculating needle with the bacteria upon it deep into solidified gelatine in a test-tube, and incubating at 20°C. for at least 1 week and some prefer 2 weeks or more. (2) B. coli coagulates milk with the production of acid when incubated at 20°C. for 48 hours or more. The formation of acid in blue litmus milk cultures is disclosed by the change to red. (3) B. coli, when cultivated in broth from which the muscle sugar has been removed, or in peptone broth, produces indol, which is indicated by a red color produced by the addi-

tion of 2 drops of concentrated sulphuric acid and 1 cc. of dilute sodium nitrite. (4) B. coli grown in a medium containing nitrates will reduce them to nitrites, free ammonia or free nitrogen gas. (5) Further information may be obtained by microscopic examination. B. coli is a short rod with rounded ends, does not form spores and loses its stain by the Gram method.

In sewages and very highly contaminated water the number of B. coli present may be exceedingly high. In drinking waters subject to suspicion the number of B. coli may be very low. It is, therefore, necessary in making tests to use varying quantities of the sample, according to the probable number of organisms present. In a highly contaminated water a small portion of the sample is diluted, as described on page 84, and a given portion of the dilution, say 1 cc., is tested for gas and plated for red colonies. A number of dilutions should be made, thus making it possible to estimate roughly the number of coli present.

Significance of B. coli.—The significance of the presence of B. coli in a water has been much discussed but even now is not as well determined as might be wished. Some have claimed that the presence of this organism is in no way characteristic of the feces of man and domestic animals, for it has been found also in the soil, in the air and in waters otherwise apparently free from contamination. The opposite view is stated by Savage in "Water Supplies" (page 141), as follows:

> "Indeed it is not too much to state that there is no evidence or observations which have ever shown that B. coli, reasonably defined, is present in any numbers in sources which have not been exposed to some form of fecal contamination."

Some maintain that this organism multiplies rapidly outside the body, but the general opinion at present is that when discharged into natural bodies of water it decreases speedily in vitality and numbers, although it may persist for several weeks and possibly for some months. Much confusion has arisen because many reports of the presence of B. coli have been based solely on the presumptive test.

In interpreting the results of tests for B. coli it is the number of these organisms found which is significant, rather than the mere fact that they are present. Prescott and Winslow ("Water Bacteriology," third edition, page 149), state that the finding of a few colon bacilli in large samples of water, or their occasional discovery in small numbers, does not necessarily have any special significance, and that the detection of B. coli in a large proportion of small samples (1 cc. or less) examined, is imperatively required as an indication of recent sewage pollution. Savage ("Water Supplies," page 157) places the following limitations to the value of the B. coli indicator:

"1. It indicates excretal (including sewage) contamination but it does not indicate the source of that pollution."[1]

"2. The indication furnished by this test as to excretal contamination may be reliable enough, but the contamination may be no longer dangerous, since B. coli can persist for considerable periods of time in soil and water, certainly longer than the typhoid bacillus.

"3. There is the great difficulty which attaches to the precise significance to be given to the bacilli which deviate from the characters ascribed to a true B. coli communis."

Another attempt at defining the meaning of the numbers of B. coli was made as follows in a progress report of the International Joint Commission (January 14, 1914, page 20), by A. J. McLaughlin, John W. F. McCullough, John A. Amyot and Frederick A. Dallyn:

"Class 1 represents those relatively pure waters found outside the zones of pollution. It is doubtful if any purer surface water than this can be found daily for long periods in inhabited areas.

Class	1	2	3	4	5
B. coli per 100 cc.	Under 2	2 to 10	10 to 20	20 to 50	Over 50
Total bacteria per cc., agar[1] (+10) 37°C.	Under 10	10 to 25	25 to 50	50 to 100	Over 100

[1] The total bacterial count on agar at 37°C. is included in the above table because of its rather definite relation to B. coli. + 10 means that the agar was sufficiently acid to require 10 cc. per liter of normal (40 g. per liter) sodium hydrate to neutralize it. This is equivalent to the Am. Pub. H. Assoc. standard of 1.0 per cent.

"Class 2 represents a slight pollution of a relatively pure water. The character, origin and intermittency of this pollution would determine its measure of safety as a drinking water. At times, such a water is undoubtedly unsafe without purification.

"Class 3 represents considerable pollution. A water belonging to this class requires unremitting care in its purification.

"Class 4 shows serious pollution. This water, in our opinion, could not be classed as a good raw water. It would impose a much more serious responsibility on a purification plant than class 2 or 3. In the Great Lakes Basin such a water should not be selected as a raw water for a purification plant; or, if the intake must be placed in such water, sources of pollution should be eliminated or nullified by sewage treatment in order to place such a water in class 2 or 3.

"Class 5. This is gross pollution. It reaches in extent from 50 to 34,000 B. coli per 100 cc. Even in the lesser numbers, such pollution with its

[1] He emphasizes the importance of this point with the illustration that a drinking water supply contaminatd with human excreta is a far more serious danger than one infected with animal excreta, for example of sheep, although both may contain equal numbers of B. coli.

fluctuations imposes an unreasonable burden upon a purification plant. Considering the source of its pollution, feces, such a water should not be considered for public use."

B. coli is not considered pathogenic for man or the lower animals under most conditions, according to Jordan ("General Bacteriology," page 262). Savage states:

"It is not B. coli itself which is objectionable, or only to a very limited degree, but that its presence points to pollution with potentially harmful matters, in particular with possible typhoid bacilli." ("Water Supplies," page 158.)

Clark and Gage state:

"While B. coli . . . cannot be considered a disease germ, nevertheless, as regards length of life under a variety of conditions, it is very similar to the bacillus of typhoid fever, and from its study in sewage effluents inferences may be drawn as to what would be the effectiveness of these filters in removing that organism." (Rept. Mass. St. Bd. Health, 1908, page 508.)

It may, therefore, be stated that if B. coli is found in large numbers the contamination is probably relatively recent and considerable, suggesting the presence of pathogenic organisms or, if absent, that they may be present later. On the other hand, it should always be borne in mind that B. coli is ordinarily present in municipal sewage in vastly greater numbers than B. typhosus, if, indeed, the latter is present at all. It is also important to remember that B. typhosus appears less hardy than B. coli, and that therefore the typhoid bacillus may be absent at the point of sampling where B. coli are found, although a companion of the colon bacilli at some prior time.

THE BACTERIUM OF TYPHOID FEVER—BACILLUS TYPHOSUS

Pathogenic bacteria, as already indicated, are those capable of producing a morbid condition. Different species cause different conditions or maladies, the typhosus bacillus, typhoid fever; the pneumococcus, pneumonia; the tubercle bacillus, tuberculosis. Jordan ("General Bacteriology," page 119) states that the conception of a pathogenic micro-organism is a relative, not an absolute one, as no microbe is known which is capable under all conditions of producing disease in all animals; that bacteria pathogenic for animals are not pathogenic for plants as a rule, and that a bacterium pathogenic for one animal species may be harmless for another. For example, typhoid bacilli when swallowed by man can produce a serious, often mortal, illness; when fed to cattle they produce no effect. He states also that as a consequence of these facts no sharp line can be drawn between pathogenic and non-pathogenic micro-organisms.

In investigations of the effect of sewage contamination of water supplies much attention has been given to so-called water-borne diseases, transmitted from patient to victim by bacteria capable of living for an appreciable length of time in natural bodies of water. Examples of such bacteria are B. typhosus, the organism of typhoid fever, the spirillum of Asiatic cholera, and probably B. dysenteriæ and other organisms causing intestinal disorders.

Characteristics of B. typhosus.—The typhoid bacillus is a cylindrical vegetable cell 1 to 4 μ long and about one-third as wide. It is capable of lively motion in liquid media, being provided with numerous long undulating flagella. Whipple states ("Typhoid Fever," page 10) that a hundred may become a billion and form colonies visible to the naked eye in 2 or 3 days when cultivated in certain artificial media. So minute, however, are these organisms that, according to Whipple, half a million of them would scarcely cover the head of a pin. The typhoid bacillus is by preference a parasite, according to Jordan, who says ("General Bacteriology," page 283) that outside the human body it has been found only in those situations where it could be more or less directly traced to an origin in the discharges of a typhoid patient or convalescent.

Viability of B. typhosus.—Danger of infection from discharges containing B. typhosus depends upon its viability and virulence. Jordan states:

"In soil and in the fecal matter of privy vaults the duration of life of the typhoid bacillus is much longer than in water. Levy and Kayser found typhoid bacilli in soil that had been manured 14 days previously with the 5-months-old contents of a vault. The evidence that any genuine multiplication can take place in the soil is not convincing, but it has been proved that the bacillus may be carried by water-currents to a considerable distance from the point where it was first introduced. Infection of wells and small water-courses is thus brought about sometimes by the washing of bacilli out of soil in which they may have lain dormant for many months. The persistence of typhoid fever around certain habitations may be plausibly explained on the supposition of an extensive soil infection. There is no doubt that the practice of using human excrement for manuring vegetable gardens entails a danger no less real because often unrecognized. The history of typhoid epidemics indicates that air-borne infection is, to say the least, exceedingly rare. Sewer air, so far as known, is never the vehicle by which the specific germ of typhoid fever is conveyed from one place to another." ("General Bacteriology," page 284.)

Typhoid bacilli rapidly decrease in numbers after discharge into fresh or salt water. Fig. 12, prepared by Whipple, indicates the length of life of the typhoid bacillus in water under natural conditions with favorable and with unfavorable environment.

While recognizing the lack of agreement in experiments, Whipple states that in very cold water the decrease in numbers is rather more rapid than in water at summer temperatures; in well oxygenated water they live longer than in stagnant water deficient in oxygen; in waters rich in organic matter the longevity is greater than in distilled water, but in nature this is more than offset by the antagonistic influence of common water bacteria and by organisms higher in the scale of life.

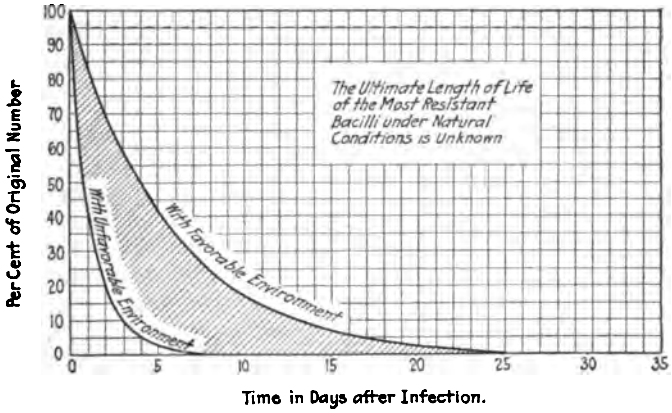


FIG. 12.—Decrease in the number of typhoid fever bacilli in water under natural conditions. (From Whipple's "Typhoid Fever," page 48.)

Decrease of B. typhosus in Sewage.—Authorities seem to agree that while typhoid bacilli die rapidly in flowing streams, some of the more resistant organisms may live for weeks and months and be carried possibly 80 miles or more, still retaining their dangerous form. In sewage, however, while ordinary bacteria multiply rapidly at first, it is believed that the pathogenic forms decrease in number.

Instructive experiments were made by Jordan with the help of parchment sacks in the following manner:

"Into the sacks was introduced a quantity of sewage, together with a strong culture of the typhoid bacilli. Sixteen such sacks were placed in the Chicago drainage canal at Robey Street and 12 at Lockport. Other sacks which were not inoculated with typhoid bacilli but contained samples of raw sewage taken from the canal at the respective points, were used for parallel experiments. Samples of water from these control sacks were plated at frequent intervals and examined as to their bacteriological content. After the culture sacks were inoculated and set in place, samples were collected from them in sterilized glass bottles, with all proper aseptic precautions, and accurately measured quantities were mingled with sufficient nutrient media of various kinds, and when the growth had sufficiently developed were examined for the identification of the typhoid bacillus.

"Taking into consideration all the experiments made in the drainage canal, the witness (Jordan) was of the opinion that the typhoid bacilli could not live for a period longer than 4 days after infection. The experiments in Illinois River led him to believe that they would not live in that stream under natural conditions for a period longer than 5 or 6 days. Summing up, the witness stated that specific infection of typhoid fever discharged into the drainage canal might live as long as 3 or 4 days after leaving the body of the pollution, but that at any rate it would perish long before Averyville (130 miles below Lockport) was reached." ("Water Supply Paper 194," page 237.)

The Metropolitan Sewerage Commission of New York, after a study of the literature, reported as follows:

"A large number of experiments have been made to test the longevity of typhoid bacilli in drinking water, but as there have also been a number of observations made upon the viability of typhoid bacilli in sewage and sea water, only these will be considered here. Giaxa found that typhoid bacilli lived for many days in sea water. Boyce and Herdman found that typhoid bacilli would live for a month in sea water. Foster and Freytag state that typhoid germs will live for a long time in sea water.

"Lawes and Andrewes found that typhoid bacilli at 20°C. would live in sterile sewage about a fortnight. Klein found that they would remain in sterile sewage 3 weeks, and if nitrates were added they existed in enormous numbers for 8 weeks. Jordan, Russell and Zeit found that typhoid bacilli lived in Chicago drainage canal water for 2 days and once for 10 days. Russell and Fuller found them alive in water in which sewage was added from 3 to 5 days. MacConkey found that typhoid bacilli could be recovered, after being introduced into crude sewage, for 6 days, but that they did not multiply and died more or less rapidly.

"There is unanimous opinion that typhoid bacilli will live in sewage whether or not the sewage be sterilized before these germs are added; that typhoid bacilli do not usually multiply in crude sewage, but retain their vitality for some days, and that typhoid bacilli may live a considerable time in sea water." (Report Metropolitan Sewerage Commission of New York, 1910, page 471.)

Fuller stated before the Franklin Institute (*Journal Franklin Inst.*, vol. clx, page 95) that the available evidence showed B. typhosus would live in unsterilized sea water in gradually decreasing numbers from about 1 week to 1 month, depending upon local conditions.

Relative Viability of B. coli and B. typhosus.—Researches by Clark and Gage into the relative viability of B. coli and B. typhosus, show the necessity of testing for the latter in many cases. They conclude:

"Comparative tests for the relative viability of B. typhosus and B. coli show us that there is a very great similarity between the length of life of the two germs under a variety of conditions. Both germs appear to follow the general laws of the removal of bacteria by sand filters, a somewhat greater

viability being noted in the case of B. coli than of B. typhosus. Most organisms of both types are destroyed quite rapidly by the effect of cold either in fluid culture or when frozen in ice. When caught in ice, both germs will live, in slowly reducing numbers, for a considerable length of time, and under these conditions the morphological and cultural characteristics of the species may lose their identity and appear to merge into a common type which there are reasons to believe would be non-pathogenic. When subjected to heat, both species appear to follow about the same rule, the majority of the typhoid germs being destroyed by 5 minutes' exposure to a temperature of 45°C. and nearly all of the B. coli being destroyed by 5 minutes at 50°C., a few individuals of each surviving exposure to temperatures up to and including 80°C. The thermal death point appears to lie between 80° and 85°C., a point somewhat higher than has usually been set for these species. Both species are rapidly destroyed by sunlight, an exposure of 30 minutes to 1 hour usually being sufficient to sterilize the culture when spread out in a thin layer. Both show the same phenomena noted in other experiments, that while the larger numbers of germs are destroyed in a very short time, a few individuals nearly always appear to be more resistant to unfavorable conditions." (Report Mass. St. Bd. Health, 1902, page 268.)

It thus appears that the two organisms have very similar characteristics, but that of the two B. coli is more resistant than B. typhosus, and hence the absence of B. coli under ordinary conditions would appear to justify the conclusion that B. typhosus is also absent.

Practicability of Isolating the Typhoid Bacillus from the Bacterial Contents of Water.—In 1906 Savage stated:

"Great as are the improvements which have taken place in the facility with which typhoid bacilli can be isolated from specifically infected excreta, with none of the different methods can it be said that the isolation of the bacillus from an infected water supply is other than a difficult and unsatisfactory procedure, and only under very favorable conditions can success be hoped for." ("Water Supplies," page 253.)

In 1908 Whipple stated:

"The isolation of the typhoid fever bacillus from infected water is a matter of such difficulty that less than a dozen authentic cases of its finding are on record. Nevertheless, theoretically, it can be done, and it is not unlikely that the future may reveal some important developments in this direction." ("Typhoid Fever," page 332.)

More recently Jackson achieved marked success with lactose bile as an enrichment medium to encourage the rapid multiplication of B. typhosus so that its presence may not be overlooked because of the greater numbers of other similar organisms present. The difficulty with many enrichment methods is that most media encourage the growth of other organisms as well, like B. coli, and that B. typhosus thus becomes masked.

In spite of recent progress in isolating B. typhosus, the process is still one of much difficulty and uncertainty. Furthermore, most questions relating to the contamination of water supplies require data not only with reference to the actual presence of B. typhosus, but also as to its probable or possible presence at some future time under the same or similar conditions. Hence the concensus of opinion at the present time with reference to problems of water supply and sewage disposal appears to be that the test for B. coli is the most satisfactory method of determining the extent of bacterial contamination and that an effort to isolate B. typhosus is not usually warranted. Undoubtedly in specific cases it will prove important to establish the presence or absence of the typhoid bacillus.

TYPHOID FEVER

Typhoid bacilli being present in great numbers in the discharges from persons suffering from typhoid fever and being capable of living and retaining their virulence during their passage through sewers and long distances in rivers, it is evident that the prevalence of typhoid fever is closely related to the problems of sewage disposal. Sewage should be disposed of and water supplies procured in such a manner as to reduce as far as practicable the danger of spreading this justly dreaded disease and others similarly communicated.

The ways in which typhoid fever is transmitted from patient to victim and the methods by which the disease may be combated are shown by Whipple in Fig. 13.

Typhoid Fever in Cities.—More or less reliable statistics regarding typhoid fever have been kept in many cities. Mortality statistics compiled by the U. S. Census Bureau furnish data from which comparisons of the prevalence of typhoid fever throughout the country may be made. From this and other sources the typhoid death rates[1] in American cities of 100,000 or more population, from 1900 to 1912, inclusive, were compiled in a report on a "Plan of Sewerage for the City of Cincinnati," presented to H. M. Waite, Chief Engineer, Department of Public Works, by Howard S. Morse, Engineer in Charge, and one of the authors. Some of these data are reproduced as Table 9.

The typhoid death rates in a few large European cities are given in Table 10. In *Engineering News*, April 21, 1910, G. R. Taylor classified American cities by their death rates per 100,000 from 1898 to 1908 as shown at the top of page 120.

[1] In mortality statistics "death rate," when applied to deaths from all causes, is generally expressed in terms of the number of deaths occurring in 1 year per 1000 persons living in the community in which the deaths occur. When applied to deaths from a specific disease, like typhoid fever, it generally means the number of deaths from the disease in question per 100,000 of persons living in the community.

In morbidity statistics "case rate" generally applies to a specific disease and should be expressed in the same terms as the corresponding death rate.

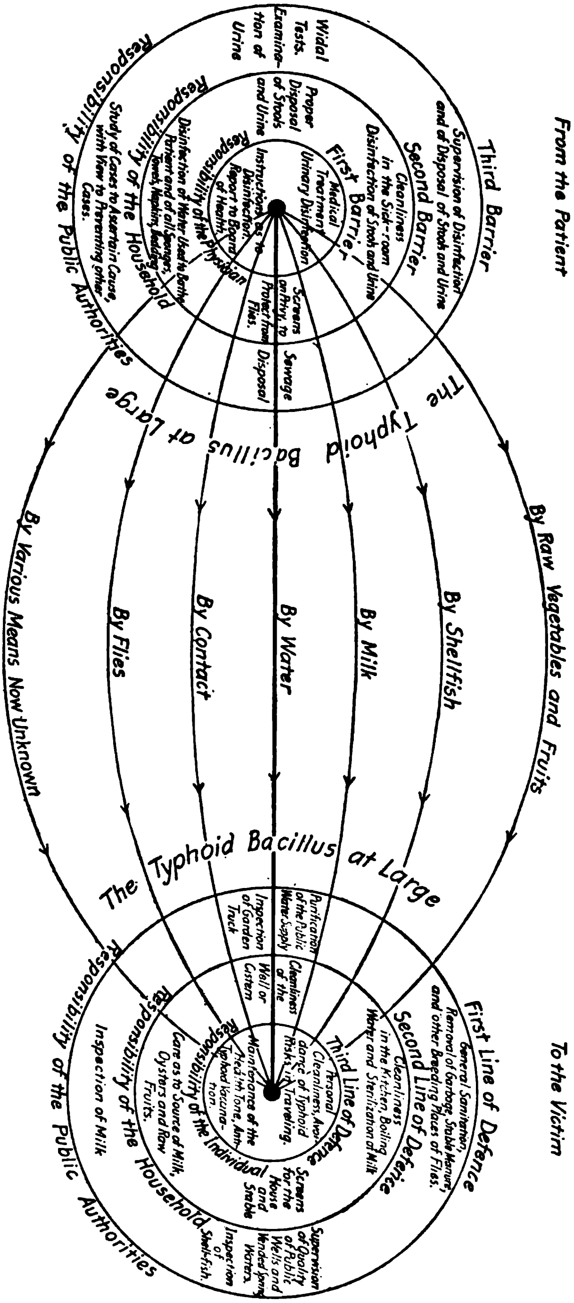


FIG. 13.—Transmission of typhoid fever and means of protection. (From Whipple's "Typhoid Fever,"— frontispiece.)

TABLE 9.—TYPHOID FEVER DEATH RATES IN CITIES OF THE UNITED STATES HAVING POPULATIONS OF 100,000 OR MORE—(*Con.*)

City	Population, 1910	Rate per 100,000 population													Average 5 years, 1908–1912	Average 13 years, 1900–1912
		1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	1911	1912		
Columbus, Ohio	181,548	53	47	36	36	141	81	35	36	102	20	18	14	19	34	53
Toledo, Ohio	168,497	41	32	35	30	38	47	47	38	42	42	37	23	32	35	37
Atlanta, Ga.	154,839	75	75	66	62	56	63	67	89	59	51	50	66	35	52	63
Oakland, Cal.	150,174	24	11	21	18	35	28	30	42	25	11	17	14	14	16	22
Worcester, Mass.	145,986	25	22	14	15	6	21	14	10	10	8	16	6	3	9	13
Syracuse, N. Y.	137,249	30	19	8	18	18	17	10	15	15	11	28	16	15	17	17
New Haven, Conn.	133,605	25	94	39	36	27	42	53	29	33	21	18	25	25	24	36
Birmingham, Ala.	132,685	.....	.....	.....	.....	.....	.....	.....	.....	83	60	50	46	40	56	.....
Memphis, Tenn.	131,105	48	53	41	44	50	37	47	44	43	49	27	65	59	49	47
Scranton, Pa.	129,867	34	34	20	18	11	17	61	75	11	16	17	14	10	14	26
Richmond, Va.	127,628	105	51	71	71	52	42	44	41	50	24	22	18	15	26	46
Paterson, N. J.	125,600	27	24	34	22	7	14	4	11	9	10	7	7	5	8	14
Omaha, Neb.	124,096	23	25	22	12	18	26	30	25	24	37	87	18	12	36	28
Fall River, Mass.	119,295	28	26	11	23	19	11	7	17	12	21	15	15	18	16	17
Dayton, Ohio	116,577	35	35	45	26	29	24	26	35	18	27	21	19	19	21	28
Grand Rapids, Mich.	112,571	31	34	51	42	62	49	38	32	31	17	28	27	31	27	36
Nashville, Tenn.	110,364	50	43	51	69	54	70	72	81	58	52	49	54	40	53	57
Lowell, Mass.	106,294	18	20	18	33	20	19	7	11	25	11	20	7	11	15	17
Cambridge, Mass.	104,839	17	9	11	11	16	12	11	8	13	8	10	3	5	8	10
Spokane, Wash.	104,402	.....	46	38	66	56	57	62	53	50	43	45	36	17	38	47
Bridgeport, Conn.	102,054	20	26	23	14	17	13	11	13	17	9	5	4	8	9	14
Albany, N. Y.	100,253	41	21	32	20	18	19	20	21	12	19	14	19	18	16	21
Average	....	38	35	36	37	36	32	35	35	27	22	25	20	16	22	30
Total	21,937,398															

Notes.—Statistics gathered from Tenth Annual Report, U. S. Mortality Statistics, 1909; supplemented with data from U. S. Mortality Statistics, 1910 and 1911; 1912 data from paper by G. A. Johnson in Proc. Am. W. W. Ass., June, 1913; and other 1912 data solicited by Metcalf & Eddy. Rates given in this table are nearest whole numbers, decimals being omitted.

TABLE 9.—TYPHOID FEVER DEATH RATES IN CITIES OF THE UNITED STATES HAVING POPULATIONS OF 100,000 OR MORE

(From Report on a Plan of Sewerage for the City of Cincinnati, pages 434, 435.)

City	Population, 1910	Rate per 100,000 population													Average 5 years, 1908–1912	Average 13 years 1900–1912
		1900	1901	1902	1903	1904	1905	1906	1907	1908	1909	1910	1911	1912		
New York, N.Y.	4,766,883	20	21	20	17	17	16	15	17	12	12	12	11	10	11	15
Chicago, Ill.	2,185,283	21	30	46	33	21	17	19	18	16	13	14	11	7	12	20
Brooklyn, N. Y.	1,634,351	25	22	24	19	22	21	16	19	14	12	12	12	10	12	18
Philadelphia, Pa.	1,549,008	37	35	47	72	55	51	74	60	35	22	18	15	12	20	41
St. Louis, Mo.	687,029	33	33	40	53	38	23	19	17	15	16	15	16	13	15	25
Boston, Mass.	670,585	24	24	22	21	24	21	21	10	25	14	11	9	8	13	18
Cleveland, Ohio	560,663	57	34	35	111	47	14	19	18	12	13	18	14	7	13	31
Baltimore, Md.	558,485	39	29	42	36	38	37	35	43	32	25	42	28	22	30	34
Pittsburgh, Pa.	533,905	144	120	136	133	136	107	141	131	49	25	28	26	6	27	83
Detroit, Mich.	465,766	28	20	24	20	18	20	21	26	20	21	23	16	17	19	21
Buffalo, N. Y.	423,715	24	27	34	35	24	24	23	29	20	24	20	25	12	20	25
San Francisco, Cal.	416,912	30	25	29	24	30	23	58	30	19	14	16	15	14	16	25
Milwaukee, Wis.	373,857	19	22	15	17	14	23	30	25	16	21	46	19	23	25	22
Cincinnati, Ohio	364,463	39	55	62	42	79	40	70	45	18	13	6	11	7	11	38
Newark, N. J.	347,469	21	24	20	23	14	14	17	23	11	12	13	11	8	11	16
New Orleans, La.	339,075	53	50	44	41	36	32	29	55	33	28	32	31	14	28	34
Washington, D. C.	331,069	80	61	79	49	47	48	52	35	39	34	23	22	23	28	45
Los Angeles, Cal.	319,198	38	31	32	38	32	27	27	22	17	16	14	12	12	14	24
Minneapolis, Minn.	301,408	41	59	27	41	40	24	33	27	19	21	59	12	11	24	32
Jersey City, N. J.	267,779	23	16	20	15	19	20	20	14	9	9	12	7	10	9	15
Kansas City, Mo.	248,381	35	44	36	74	39	55	33	34	29	29	54	30	12	31	39
Seattle, Wash.	237,194	35	28	27	29	27	25	24	48	18	24	14	10	8	15	24
Indianapolis, Ind.	233,650	41	33	44	52	70	31	41	31	28	22	29	26	18	25	36
Providence, R. I.	224,326	24	25	21	20	16	20	19	8	17	11	18	12	10	14	17
Louisville, Ky.	223,928	64	46	61	61	63	51	71	71	47	45	32	24	19	33	50
Rochester, N. Y.	218,149	17	17	12	12	16	12	17	16	11	9	14	11	12	11	14
St. Paul, Minn.	214,744	22	14	14	10	14	11	21	18	13	19	20	11	11	15	15
Denver, Col.	213,381	40	47	56	50	26	34	56	83	42	24	28	18	13	25	40
Portland, Ore.	207,214	35	20	31	29	20	25	29	22	23	22	22	19	17	21	24

Range.......	0/10	10/20	20/30	30/40	40/50	50/60	60/70	70/100	100+
Cities, no....	15	97	84	64	50	36	19	24	10

Ratio of Cases to Deaths from Typhoid Fever.—Whipple stated (1911) that he was "pretty well convinced that under ordinary conditions the ratio between cases and deaths is considerably more than 10 to 1. Wherever house-to-house canvasses have been made it has not been uncommon to find from 12 to 15 cases for every death." In another place he said:

TABLE 10.—TYPHOID FEVER DEATH RATES PER 100,000 POPULATION IN EUROPEAN CITIES

Year	London	Paris	Berlin	Vienna	Dresden
	Population for 1910				
	7,250,000[2]	2,750,000[2]	2,000,000[2]	2,000,000[2]	550,000[2]
1898............	13.0[1]	9.9[1]	4.3[1]	6.8[1]	4.3[1]
1899............	17.8[1]	29.0[1]	4.1[1]	4.1[1]	7.3[1]
1900............	16.5[1]	34.6[1]	5.8[1]	8.3[1]	4.1[1]
1901............	12.1[1]	13.7[1]	4.7[1]	4.5[1]	6.4[1]
1902............	12.8[1]	....	2.7[1]	3.0[1]	4.0[1]
1903............	8.6[1]	10.4[1]	3.2[1]	3.9[1]	5.3[1]
1904............	6.2[1]	12.2[1]	3.7[1]	3.4[1]	2.8[1]
1905............	5.2[1]	....	5.3[1]	4.4[1]	3.7[1]
1906............	6.0[1]	11.0[1]	4.0[1]	5.0[1]	7.0[1]
1907............	4.0[1]	10.0[1]	4.0[1]	3.0[1]	2.0[1]
1908............	5.0[3]	....	4.0[3]	4.0[3]	6.0[3]
1909............	2.2[3]	8.4[3]	4.2[3]	2.8[3]	4.2[3]
1910............	3.3[3]	5.6[3]	2.9[3]	3.8[3]	2.2[3]

[1] From Report of Metropolitan Sewerage Commission of New York, 1910, page 469.
[2] From A. J. McLaughlin's paper on "The Eradication of Typhoid Fever," Boston Medical and Surgical Jour., May 23, 1912.
[3] A. J. McLaughlin's paper on "The Necessity for Safe Water Supplies in the Control of Typhoid Fever," Reprint 76 of Public Health Reports, Public Health and Marine-Hospital Service of the United States, 1912.

"The incompleteness of morbidity statistics of typhoid fever based on physicians' reports, is usually so great as to make them practically useless. A few examples will suffice to illustrate this:

"In Brooklyn, N. Y., in 1904 there were 1050 reported cases of typhoid fever and 303 deaths, indicating a fatality of 29 per cent.; in New York City (Borough of Manhattan) in the same year there were 2136 cases and 309 deaths, indicating a fatality of 14.5 per cent. When complete statistics are obtained, the percentage fatality in typhoid fever almost never exceeds 10 per cent. In Waterville and Augusta, Me., where in 1902–03 statistics were obtained in a house-to-house canvass, the fatality was 8.6 per cent.; in Ithaca in 1903 it was 6.1 per cent.; in the United States military camps

during the Spanish war it was 7.6. The fatality varies according to age, as referred to elsewhere.

"When an epidemic occurs, physicians are more likely to report cases of typhoid fever than at other times when there is no excitement. Thus, in Cleveland, the ratio of typhoid deaths to reported cases in 1902 was 27 per cent.; during the epidemic years of 1903–04 it was 14.5 per cent.; in 1905, after the excitement had subsided, it rose to 33 per cent. It is not infrequent that health department records will show a larger number of deaths from typhoid fever than there were cases reported." ("Typhoid Fever," page 99.)

Seasonal Distribution of Typhoid Fever.—The seasonal distribution of typhoid fever in the United States is shown by Fig. 14 from the 1900

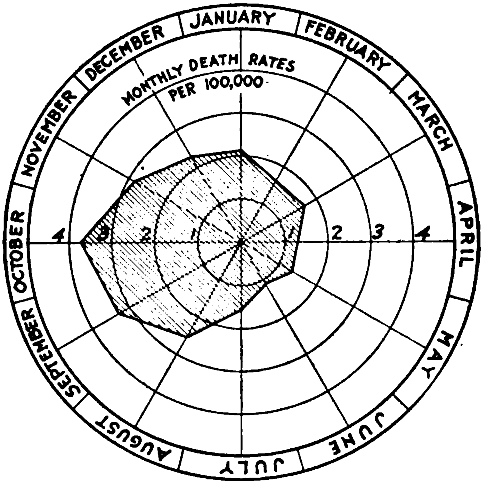


FIG. 14.—Seasonal distribution of typhoid fever in the United States. (From Whipple's "Typhoid Fever," page 121.)

report of the U. S. Census Bureau. Water-borne typhoid is more prevalent during winter than at other seasons. In cities with contaminated water supplies, the normal curve does not apply, as illustrated by Whipple with Fig. 15. The authors found in Cincinnati that the number of cases of typhoid prior to the filtration of the water supply was highest in the winter,[1] and after filtration highest during the summer and fall.

In cities with uncontaminated water supplies the greater prevalence

[1] "These high death rates in the cooler months were at first puzzling, and such instances have obscured the usual seasonal variations and led some sanitarians to deny the existence of any relation between typhoid fever and temperature. As soon, however, as it appeared that curves of this type were associated with polluted water supplies, their significance became clear. The fall and spring epidemics coincide with the fall rains and the spring thaws, which wash infecting material from vaults on the banks, or through storm overflows, into the water supplies, carrying it fresh and virulent to the consumers below." (Sedgwick and Winslow, *Jour. N. E. W. W. Assoc.*, vol. xx, 1906, page 51.)

TABLE 11.—SUMMARY OF TYPHOID FEVER EPIDEMICS, STUDIED AND FOUND TO HAVE BEEN CAUSED BY THE CONTAMINATION OF WATER SUPPLIES BY SEWAGE

(Compiled from "Typhoid Fever," by Geo. C. Whipple, 1908)

Place	Year	Approx. No. of cases	Causes and remarks
Lansen, Switzerland	1872	130	Pollution of spring by underground connection with contaminated brook.
Plymouth, Pa......	1885	1104	Pollution of water supply by typhoid patient.
Lowell, Mass.......	1890–1891	550	Pollution of Merrimack Riv. used as source of water supply by cities above.
Lawrence, Mass.....	1890–1891	....	Pollution of Merrimack Riv. by infected sewage of Lowell.
Newport, R. I......	1900	80	Pollution of well by underground connection with contaminated water.
New Haven, Conn..	1901	514	Pollution of water supply by typhoid patient.
Baraboo, Wis.......	1901	....	Polluted water from flume, pumped into supply on account of defective valves.
Waterville, Me.....	1902	....	Pollution of Messalonskee Riv. by sewage of communities above.
Augusta, Me.......	1902	....	Pollution of Messalonskee Riv. by infected sewage of Waterville, above.
Auxerre, France....	1902	....	Pollution of collecting gallery by underground connection with contaminated Yonne River.
Ithaca, N. Y.......	1903	1350	Pollution of water supply by sewers 5 miles above.
Butler, Pa..........	1903	1270	Pollution of creek by typhoid patient; by-passing of city filter allowed typhoid bacilli to enter water supply.
Lowell, Mass.......	1903	168	Admission of river water to mains through defective check valves.
Cleveland, Ohio.....	1903–1904	....	Pollution of Lake Erie by sewage of city.
Millinocket, Me.....	1904	200	Pumping of polluted Penobscot Riv. water into water supply mains during fire.
Mt. Savage, Md....	1904	120	Pollution of spring by sewage filtering through soil.
Basingstroke, England.	1905	164	Pollution of well by underground connection with contaminated water.
Scranton, Pa.......	1906	1155	Pollution of reservoir by sewers above.
Trenton, N. J.......	1907	90	Pollution of well by sewage filtering through soil for 300 ft.
Chicago, Ill........	Previous to completion of Drainage Canal	....	Pollution of Lake Michigan, the source of the water supply. Frequent epidemics.
Pittsburgh and Allegheny, Pa.	Previous to 1908	5000 cases annually	Pollution of Monongahela and Allegheny Rivers, used untreated as water supply.
Burlington, Vt......	Previous to 1908	....	Pollution of Lake Champlain used untreated as water supply. Frequent epidemics.

of typhoid during the late summer and fall is apparently due to the facts that the common house fly is most numerous and active during

August, September and October, that the disease is contracted by persons away from home on vacations, where greater opportunities of infection frequently exist, and that uncooked fruits, vegetables and some other foods are eaten more abundantly during this season.

Transmission of Typhoid by Water Supplies.—The contamination of municipal water supplies by sewage is likely to result in typhoid epidemics because of the large number of persons exposed to the danger of infection. A summary of epidemics traced to this cause, compiled from data in Whipple's "Typhoid Fever," is presented in Table 11.

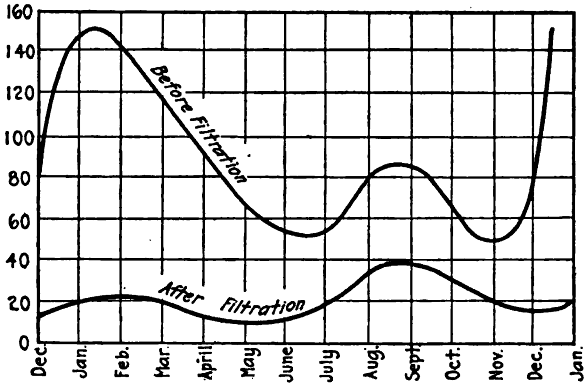


Fig. 15.—Seasonal distribution of typhoid fever at Albany, N. Y., before and after filtration of water supply. (From Whipple's "Typhoid Fever," page 123.)

Transmission of Typhoid by Shellfish.—It has been clearly established that the growing or fattening of shellfish in sewage-contaminated waters may lead to the infection of consumers. A number of typhoid epidemics traced to eating contaminated shellfish as the most probable cause of infection is listed in Table 12. Danger of infection is not limited to raw shellfish, for cooking does not always subject them to a high temperature for sufficient time to kill all bacteria. Sterilizing the shellfish by sufficient cooking often makes them unpalatable.

Laws for the Prevention of Transmission of Typhoid by Shellfish.—Several states have passed laws to prevent the public from consuming infected shellfish, and the U. S. Board of Food and Drug Inspection has made decisions to restrict the interstate distribution of contaminated shellfish.

In Massachusetts, the State Department of Health is empowered in Chapter 91 of the Revised Laws (1902) to prevent taking shellfish from contaminated waters. The law reads:

"*Section* 113.—The State Board of Health may examine all complaints which may be brought to its notice, relative to the contamination of tidal

TABLE 12.—SUMMARY OF TYPHOID FEVER EPIDEMICS THOUGHT TO HAVE BEEN DEFINITELY TRACED TO POLLUTED SHELLFISH

(From report of Metropolitan Sewerage Commission of New York, 1910, pages 481 to 483)

Place	Year	Approx. No. of cases	Causes and remarks
Wesleyan University	1894	23	Oysters dredged from deep water of Long Island Sound; fattened by placing in brackish water 300 ft. below private sewer outlet. Two cases typhoid fever in house drained by this sewer.
Southend-on-Sea, England	......	105	50 per cent. at least attributed to consumption of contaminated shellfish.
Yare, England	......	....	Typhoid endemic. Cases diminished 30 per cent. in 1901 after sale of mussels was prohibited.
Brighton, England	1894 to 1902	643	37 per cent. ascribed to shellfish from layings exposed to sewage contamination. Typhoid fever found to exist in population draining to vicinity of layings.
Manchester, England	1904	....	Niven, before R. C. S. D., attributes number of cases to polluted shellfish.
London	1902	....	8 per cent. of cases estimated to be due to eating of shellfish.
Lawrence, Long Island	1905	31	21 caused by eating or handling sewage contaminated oysters.
Winchester-Southampton, England.	1902	21	Raw contaminated oysters eaten at banquet.

waters and flats in this commonwealth by sewage or other causes; may determine, as nearly as may be, the bounds of such contamination, and if necessary, mark such bounds. It may also, in writing, request the Commissioners on Fisheries and Game to prohibit the taking from such contaminated waters and flats of any oysters, clams, quahaugs and scallops. Upon receipt of such request, said commissioners shall prohibit the taking of such shellfish from such contaminated waters or flats for such period of time as the State Board of Health may prescribe.

"*Section* 114.—Whoever takes any oysters, clams, quahaugs, or scallops from tidal waters or flats from which the taking has been prohibited, as provided in the preceding section, shall forfeit not less than $5 nor more than $10 for the first offence, and not less than $50 nor more than $100 for each subsequent offence; but such penalties shall not be incurred until one week after the Commissioners on Fisheries and Game shall have caused notice of such prohibition, with a description, or the bounds, of the tidal waters or flats to which such prohibition applies, to be published in a newspaper published in the town or county in which or adjacent to which the tidal waters or flats to which such prohibition applies are situated."

This statute was modified by Chapter 285, Acts of 1907, as follows:

"*Section* 1.—Whenever at the request of the State Board of Health, under provision of Section 113, Chapter 91 of the Revised Laws, the Commissioners

on Fisheries and Game have prohibited or may hereafter prohibit the taking from contaminated waters or flats in any city or town of any clams or quahaugs, the Board of Health of such city or town may grant permits in writing to any person to take from such waters clams and quahaugs to be used for bait only and in such quantities and upon such conditions as they shall express in their permit."

In some states an effort has been made to protect the shellfish industry as well as to safeguard the public health. Rhode Island apparently intended this in Chapter 577, Acts of 1910, the important sections of which are the following:

"*Section* 1.—No person shall deposit in, or allow to escape into, or shall cause or permit to be deposited in, or allowed to escape into, any of the public waters of this state, any substance which shall in any manner injuriously affect the growth or sale of the shellfish in or under said waters, or which shall in any manner affect the flavor or odor of such shellfish so as to injuriously affect the sale thereof, or which shall cause any injury to the public and private fisheries of this state.

"*Section* 7.—The Commissioners of Shellfisheries shall inspect the premises designated in Section 8 of this chapter at such times as they may deem advisable, for the purpose of determining whether said premises are kept in a proper sanitary condition for opening, handling, or packing shellfish for the trade. Also said commissioners shall inspect the methods followed on the premises in opening, packing, or preparing shellfish for the trade, to determine whether such methods are proper from a sanitary standpoint.

"*Section* 8.—The premises which come within the scope of this chapter are all establishments where oysters or other shellfish are opened, packed, or prepared for the trade. Retail or wholesale markets where shellfish are sold which were purchased from the original opening or packing houses designated in this section shall not come within the scope of this chapter.

"*Section* 9.—Said commissioners shall inspect any or all the leased oyster grounds and other shellfish grounds within the state at such times as they may deem advisable, to determine whether said grounds are in a proper sanitary condition for the production of shellfish for consumption as food.

"*Section* 12.—No person shall take shellfish from any grounds which are not certified by said commissioners as being in a sanitary condition, except for the purpose of transplantation. No person shall prepare shellfish for the trade except on premises and by methods certified by said commissioners as being sanitary.

Transmission of Typhoid by House Flies.—It has been proved that transmission of typhoid by flies is possible (Whipple's "Typhoid Fever," page 67). Infected sewage discharged into bodies of water may be washed upon the shores and otherwise become accessible to the fly. Jackson found floating sewage matter in New York docks a source of danger. The most common danger is the open privy vault in close proximity to the house. The infection of food by flies from such privies

is to be feared, and one means of protection is to abandon such privies, a natural sequence to the construction of sewers.

It is difficult to demonstrate the extent to which increased sewerage facilities reduce disease and improve the health of the community. An investigation by J. B. F. Breed, Chief Eng., Commissioners of Sewerage, Louisville, Ky., in 1912, furnished significant data, Table 13, although perhaps not sufficient for positive conclusions upon one phase of this subject.

"It is interesting to note that there has been a gradual decrease in the death rate from typhoid fever with the gradual increase in the number of house connections (with sewer) per 1000 population. While the water filters were put into operation in August, 1909, they could not have had any effect upon the rapidly decreasing death rate before that time, as the statistical year closed August 31. A considerable part of this continued decrease in the typhoid death rate is undoubtedly due to the improved water supply, although it is true that the relation existing between the number of house connections and the typhoid fever death rate during the years previous to the filtration of the municipal water supply has continued.

"Further light may be thrown upon the cause of typhoid by an examination of the data contained in the report of the Health Department upon the investigation of typhoid fever causes for the year ending August 31, 1911. During this year there were 138 cases which may be subdivided as shown in Table 14.

TABLE 14.—TYPHOID FEVER CASES FOR YEAR ENDING ABOUT AUGUST 31, 1911, LOUISVILLE, KY.

Cases contracted outside city	5	3.6 per cent.
Cases drinking well, spring or cistern water	24	17.4 per cent.
Cases drinking filtered river water	109	79.0 per cent.
Total	138	100.0 per cent.
Cases using open privy vaults	101	73.0 per cent.

"There were 5 cases reported to have been undoubtedly contracted outside the city. Of the remainder, only 24, or 17.4 per cent., of the typhoid cases, were found to have used well, spring or cistern water, while 109, or 79 per cent., were supplied with the filtered river water. It thus appears that only a moderate proportion of the cases could be attributed to the use of well water. One hundred one, or 73 per cent. of the typhoid cases, were found in families using open privy vaults, and while it is not possible to prove that any of these cases were contracted through the agency of flies and these vaults, nevertheless the large number of cases occurring in localities where open vaults are in common use is very significant, and leads to the conclusion that the improved sewerage facilities have been an important factor in the gradual reduction of the typhoid fever death rate during the last 30 years." (Final report, Commissioners of Sewerage, Louisville, 1913, page 66.)

TABLE 13.—POPULATION, MILEAGE OF SEWERS, NUMBER OF HOUSE CONNECTIONS AND TYPHOID FEVER RATE, 1880 TO 1912 INCLUSIVE, LOUISVILLE, KY.

Year	Population	Total mileage of sewers per 100,000 population	Total number of house connections per 1000 population	Deaths from typhoid fever per 100,000 population
1880	123,151	30.1	11.51	66
1881	126,000	29.8	12.03	107
1882	129,500	29.3	12.49	92
1883	132,500	31.5	12.81	44
1884	136,000	35.9	13.94	114
1885	140,000	34.9	14.90	109
1886	144,000	34.2	15.43	81
1887	148,000	33.4	16.10	81
1888	152,500	32.7	16.70	87
1889	156,500	35.0	17.85	92
1890	161,129	36.8	18.95	88
1891	165,000	37.4	20.23	78
1892	170,000	37.8	21.25	68
1893	174,500	38.3	22.40	77
1894	179,000	39.2	24.40	81
1895	183,000	40.9	27.40	69
1896	187,000	42.0	29.55	70
1897	192,000	43.8	31.10	48
1898	196,000	44.5	33.20	64
1899	200,000	46.6	37.62	66
1900	204,731	48.7	39.70	52
1901	208,000	50.0	41.40	50
1902	212,000	50.6	44.20	51
1903	215,000	51.0	47.60	73
1904	219,000	50.9	49.60	51
1905	222,500	50.5	51.40	51
1906	226,000	50.5	53.50	61
1907	230,000	51.0	54.35	74
1908	235,500	54.1	57.45	46
1909	237,000	59.4	63.50	40
1910	241,000	64.9	71.70	29
1911	244,500	67.5	78.30	24
1912	247,758	71.7	86.00	19

CHAPTER IV

PLANKTON

The biological processes going on in streams, ponds, lakes and the sea are no less important than those occurring in various kinds of artificial sewage filters. The action of the artificial filters and beds is believed to be dependent largely, though not wholly, upon bacterial action, whereas the changes wrought upon sewage discharged into natural waters are the results of the life processes not only of bacteria but also of higher organisms, some of which are visible to the naked eye but many of which can be distinguished only with the aid of a magnifying glass or microscope. The character of such changes and the kind of organisms predominating at the time are dependent largely upon the nature of the substances in the water. Sewage and the effluents of sewage treatment plants contain large quantities of organic matter, some simple and some complex, and some inorganic matter like nitrates, free ammonia, carbon dioxide, hydrogen sulphide and other substances. These constitute food for a great number of micro-organisms as well as for bacteria, and the changes brought about by the agency of these low forms of life are now recognized as of great importance in maintaining natural bodies of water in a wholesome and clean condition. A discussion of the problems involved in the disposal of sewage would, therefore, be incomplete without direct reference to the part played by the micro-organisms in the ultimate purification of sewage in natural waters.

In 1887 Victor Hensen described a method of studying the minute floating organisms of lakes and oceans, to which he gave the name "plankton." Later Dr. Maximilian Marsson defined the word as meaning "all living matter which drifts involuntarily," and Prof. George C. Whipple ("Microscopy of Drinking Water") states that the term "plankton" is practically synonymous with the term "microscopic organisms" of the sanitary biologist.

The plankton is composed of both plant and animal life, known as phytoplankton and zooplankton respectively. Its relation to sewage disposal and the self-purification of streams has been studied by German investigators, particularly by Marsson and Dr. R. Kolkwitz of the Royal Testing Station for Water Supply and Sewage Disposal at Berlin. The following abstract from *Engineering News* (vol. lxvi, 1911, page 246), is from Emil Kuichling's translation of a lecture by Marsson (*Mit. Kön.*

Prüf. Ans. f. Was. u. Abw., vol. xiv, 1911) summarizing his views on the subject:

"I have called attention particularly to the plankton, because its vegetable component is the fundamental food supply or condition of existence for all aquatic life. It comes from the products of the decomposition of the albumen which finds its way into the water from decaying animals and plants as well as from sewage. The self-purifying power of natural waters is merely the maintenance of the proper equilibrium between retrogressive and progressive metamorphosis, and frequent reference thereto will be made in the following:

"In discriminating between vegetable and animal plankton, also taking into account the remaining aquatic organisms that are found on the bottom and shores, we find that we must divide them into two classes, viz., food producers and food consumers. The plants which assimilate inorganic matter and build up organic compounds by means of their chromophyll coloring matter belong to the first class; and all other organisms, such as microscopic protozoa, rotifera, etc., together with the larger ones up to the fishes, belong to the second class. or food consumers.

VEGETABLE PLANKTON

Food Producers.—"Let us now consider the plants somewhat more closely. In waters the algæ play the most important part, as they exhibit the greatest diversity of form. They are found in all places where water has collected, even in long-stagnant rain water, but their vegetation varies with the seasons and the character of the water. While the plankton contains many unicellular forms and coherent colonies, or strings of such, we find in shallow waters, streams, ditches, and shores mostly the confervæ, or thread-like algæ, which appear as thick mats or strands of light or dark green color. The blue-green group called schizophyceæ is also widely distributed and representatives thereof usually produce the so-called "blooming" of our rivers and lakes, while others, especially the oscillatoria, often settle in strongly polluted water-courses, such as the gutters and ditches of unsewered towns and the drainage channels of barnyards. When they form a covering of wet and polluted ground, they usually have a deep black color, but similar kinds are sometimes red and have been mistaken for dried pools of blood.

"The other groups of algæ have only a few representatives in fresh water. Of these the diatoms and bacillariaceæ have been most extensively studied by microscopists, as they are found in all seasons in every water-course and pool from the pole to the equator, and as well in icy streams from glaciers as in hot springs. They sometimes appear in such enormous quantities as to render a counting of individuals impracticable. It was formerly believed that in winter our frozen rivers and lakes were devoid of life, as the water was then unusually clear, but on closer examination by filtering the water through a net of fine silk gauze or by killing the living organisms with alcohol, formalin, etc., and concentrating them by sedimentation or other means

an abundance of microscopic organisms of various kinds, and especially diatoms, will be found in all seasons. The cell body of these algæ is held together by a silicious skeleton, which has often so delicate a structure as to be defined clearly only by the best microscopes; and hence certain species of diatoms, such as Surirella gemma and Pleurosigma angulatum, are used as test objects for the best objectives. Through their brown coloring matter called diatomin these silicious algæ are as capable of assimilating organic matter as are the green and blue-green algæ by means of their normal green coloring matter called chlorophyll.

"As the assimilative activity of the algæ is a very important factor in maintaining the purity of our streams, it is necessary for us to consider briefly the structure of the microscopic plant cells, of which the higher plants are composed.

"In opposition to the animal cell, a well-defined vegetable cell is enclosed by a dense skin called the membrane. For this reason the cells of a plant are sharply separated from each other. Each such cell possesses a much greater independence than the cells of the body of an animal, or at least of the higher developed animals. In almost every plant cell there is a nucleus, the remainder of the volume being filled by the cell plasm or cytoplasm, which is rich in albuminous substances, and in which the carriers of coloring matter, or the chromatophores, are distributed. These three components (nucleus, cytoplasm and chromatophores) taken together are commonly called protoplasm, or more briefly plasm, which includes all the living constituents or protoplasts of the cell. The chief function of the cell is based upon a reciprocal action between the nucleus and the plasm. The chromatophores or chloroplasts, however, whose predominant green coloring matter is chlorophyll, exercise an important function in the cell, since with the help of the vibrations of light they are enabled to dissociate carbon from the carbonic acid that is contained in air and water, and to prepare with the elements of water organic matter having little oxygen. Polymeric carbonaceous products are thus formed which then develop into starch and certain kinds of sugar; and these compounds, in conjunction with the simplest combination of nitrogen, finally serve to form albumen and other complex combinations for the use of plants, whereby the weight of dry organic matter in the latter is increased. The nitrogen required to form the albuminous compounds is obtained from ammonia and nitrates, and other elements found in the ground and water also appear to play a not less important part in the formation of albumen.

"**Assimilation.**—This entire synthetic process in which oxygen is dissociated from carbonic acid in the form of gas, is called assimilation. . . .

"It had previously been found that in addition to the matter obtained by assimilation, diatoms could absorb albuminous or similar nitrogenous compounds from the medium surrounding them, and hence that they derive their food from two distinct sources. Subsequent physiological experiments showed that diatoms as well as green algæ can absorb not only carbon compounds from dissolved organic matter, but also organic nitrogen; and that when carbonic acid is rigidly excluded from the water they can digest diluted volatile fatty acids, amido-acids, skatol, urea, peptone, and other substances. Quite recently it was proved that certain diatoms, especially some of the

marine varieties, were entirely devoid of chromophyll, or the coloring matter required for assimilation; that they subsisted exclusively on the products of decomposition derived from their surroundings, without recourse to assimilation; and hence that they needed only a medium containing a sufficient supply of food of various kinds. Such algæ have accordingly become true saprophytes, or residents of putrefying matter, and by giving up their chromophyll they have taken the last step from a mixotropic to a heterotropic nourishment.

"This direct absorption of dissolved organic matter by algæ is of the utmost importance for the purification of streams by eliminating therefrom the soluble products of putrefaction. As all the algæ that are held in suspension in the water of a river, as well as those that are lodged on the bed and shores, are busily engaged day and night in the work of producing fresh albumen, starch, and indirectly fats, from animal and vegetable refuse matter, it follows naturally that numerous animal organisms will develop in the water to make use of these products and convert them into living flesh.

"Through these various investigations and the cyclical transformation of matter, it has become possible to understand how a constant biological self-purification takes place in our streams and lakes, even during the winter and under the ice, and that the maintenance of the plant life insures the continuation of the animal life in the water.

"**Molds or Fungi.**—A similar mode of nourishment is exhibited by the molds or fungi. These plants cannot assimilate inorganic matter like those having chlorophyll, but must rely for their food on the presence of other organisms or of suitably prepared organic matter. They represent the realm of darkness and receive the penalty which nature has placed on the giving up of independent nutrition; they are either parasites or saprophytes, that is to say, they subsist on the products of the decomposition of dead organisms and vegetable and animal refuse matter, such as is contained in the sewage of cities and the wastes of factories. . . .

"**Boundary Line between Plants and Animals.**—In the foregoing much consideration has been given to the aquatic food producers, whose significance in maintaining the purity of our streams has not been recognized sufficiently up to the present time and we have also become convinced that the animal world with its innumerable species is dependent on the synthetic work performed by plants. No animal is capable of making, like the plant, the food required for its maintenance out of inorganic matter; and this fact affords a marked distinction between plants and animals, or between food producers and food consumers. The boundary lines that were formerly drawn between the animal and vegetable kingdoms have been obliterated by recent investigations. There are, however, many organisms of doubtful character, in which the peculiarities of both kingdoms are more or less mixed and blended and which are therefore claimed by both botanists and zoologists. Such organisms possess the fundamental properties of living protoplasm, namely sensation and motion, which are fully developed in the higher stages of animal life, while in plants they are manifested only in exceptional cases.

"The green flagellates, which move by means of their flagella, must be

regarded as plants, as they excrete oxygen by means of their chloroplasts. This peculiarity does not belong to any animal.

"Among these flagellates the euglenas are of great importance in the transformation of putrescent matter. They not only absorb such matter directly, but they also produce oxygen while in the sunlight and thus oxidize compounds that have been reduced by putrefactive fermentation, as well as offensive gases. The euglenas and their relatives are therefore found mainly in nauseous pools whose surface they cover with a green scum. They appear on every sewage farm and especially on the surface of sewage held in tanks. They also develop in great numbers in sluggish indentations of the banks of polluted streams from which they are carried along by the current. When found plentifully in the plankton, they afford proof that the water has been contaminated. . . .

ANIMAL PLANKTON

"**Food Consumers; Bacteria Eaters.**—Let us next take up the consideration of the most important groups of animals which help to purify polluted water. Among the protozoa, which in comparison with the metazoa have the morphological value of a single cell, the colorless flagellated infusoria may be cited first as food consumers. By means of their small flagella they direct minute particles of food, bacteria, etc., into their mouth-vacuoles which retain them. The numerous species of monads are thus typical bacteria-eaters, and in stagnant, putrefying water they often develop in extraordinary numbers. There are also many kinds of ciliated infusoria that eat bacteria. They are somewhat more highly organized than the preceding class, and usually have a mouth aperture with a passageway to the interior. With their cilia or their undulating membranes, they produce whirling currents in the surrounding water containing bacteria and other minute particles of food, including the flagellates just mentioned, whereby such bodies are brought into their vacuoles. These relatively large unicellular organisms thus dominate over the other living organisms in their vicinity, according to their size, like a cannibal among fish. Many of the metazoa, including the rotatoria, are likewise bacteria-eaters and obtain their food in the same manner as the ciliates. Besides bacteria, this food material embraces other bacteria-eaters, diatoms, small algæ, etc. If many bacteria-eaters are found in a water, it can safely be inferred that many bacteria are present, and hence also a quantity of putrefying substances, thus rendering a tedious counting of germs wholly unnecessary before expressing an opinion as to the sanitary quality of the water.

"A great horde of lower animals is, therefore, striving incessantly to destroy the bacteria. If the latter disappear, the bacteria-eaters will also vanish, as they cannot find adequate subsistence in the purer water, and must either starve or become the prey of other animals. Heretofore it was believed that the bacteria died gradually and were subject to a process of sedimentation; but if such were the fact, the beds of our streams would present a very different appearance. It is also highly improbable that an actual settlement of such extremely small organisms can take place in a

stream having even a moderate current. Obviously some of the bacteria will be dragged to the bottom along with the heavy matter in suspension, but the great majority will not settle. Experiments in this direction have been made for improving the condition of the water in large aquaria and fish ponds by the precipitation of clay; but few have thought of the natural biological processes of water purification, because an unexpectedly extensive view of these processes has been gained only quite recently by planktological and thorough hydrobiological investigations.

"**Other Food Consumers.**—Among the organisms that are carried in suspension in water, the minute crustacea must also be considered. They are found in all seasons in the plankton of rivers, although in less quantity in swiftly flowing streams. At one time the cladocera, or water fleas, will predominate, while at another time the copepoda will be the more numerous, according to their cycle of propagation. Their significance for the purification of water has likewise not been sufficiently appreciated. Concerning the quality of their food, numerous but not conclusive observations have been made. Some investigators assert that they live only on animal food, such as protozoa, while others claim that they use only vegetable food, such as diatoms and other minute algæ; but it must be remembered that both protozoa and minute algæ usually adhere to decaying detritus of animal and vegetable origin. . . .

"**Food for Fish.**—In addition to the bacteria-eating and omnivorous animals cited, there are many other species, especially rotatoria and large ciliata, which feed preferably on algæ, as their digestive cavities are frequently found stuffed with green algæ and brown diatoms. All of these minute animals and particularly the small crustacea are of importance as food for fishes and chiefly for young fishes. After consuming the contents of their egg sacks, the latter take for their first food the very small animalcules such as rotatoria and young crustacea; as their size increases, they feed on full-grown crustacea, and so on until they finally become the prey of larger fishes. The plankton thus forms the beginning of the cyclical course of matter for a fish. Carp can feed exclusively on plankton crustacea until they attain a weight of 1 lb. The putrescent matter that finds its way into our waters is thus transformed into useful fish flesh and from the relations of the interchange of matter between plant and animal it becomes evident that a certain degree of pollution of a stream is necessary from the point of view of social economy.

"In the plankton of ponds, the animal components usually preponderate over the plants; hence we find that the slower the current of a stream, the more closely does its plankton agree in composition with that of a pond and the algæ that cause "water blooming" are then also found more plentifully. In rapidly flowing streams, on the other hand, all forms of organisms are less abundant, and the maintenance of the purity of the water depends more on the attendant physical factors than on the plankton. This is confirmed by all the examinations of streams that I have made in various parts of Germany; and especially was such the case in the Rhine which contains little plankton, and some of its tributaries, also in the Danube and other similar large rivers, in contradistinction to the more sluggish rivers

Spree and Dahme, the latter of which is very rich in plankton and decomposable organic matter throughout the entire year.

"It may also be said that the greater the quantity of impurities reaching a stream, the stronger will be the development of the plankton and the greater the amount of work it will have to perform by means of its nutrition and growth. Nature always endeavors to help herself. The supply of fresh matter and organisms is obviously essential to the renewal of the plankton of a stream and both of these are derived from coves, abandoned channels, and tributary ponds and lakes, many of which occur in the drainage areas of the rivers Spree and Dahme, for example. In the plankton of the waters of the Zoological Garden of Berlin, which are diverted from and returned into the Spree, I found in a single year more than 200 species of organisms, which led me to describe these park waters as culture tanks for numerous kinds of algæ that are capable of keeping the water clean throughout the entire year. . . .

TRADE-WASTES DANGERS

"Concerning the capacity of a stream to digest or purify sewage and other wastes, definite figures and limits, such as those[1] given by Pettenkofer, cannot be made readily. The maximum quantity of such liquid that a stream can digest depends not only on its physical characteristics, but also on its individuality in other respects. Both the biological and the chemical factors must here be taken into consideration. If the industrial wastes contain much acid, alkali or other matter that is poisonous to the animal and plant life of a stream, entirely different conditions are presented from those which occur with domestic sewage and harmless wastes. Thus it has been found that tar products have destroyed all the animal and plant life in the mud bottom of long stretches of river, and that alkalies and acids have killed off every kind of fish by attacking the gills and thereby causing death by suffocation. With a proper quantity of harmless organic wastes, on the other hand, the microscopic organisms suspended in the water, as well as the larger flora and fauna, will labor incessantly to keep the stream in a healthy condition.

"In regard to injury to fishes the opinion has hitherto been that the numerous kinds of industrial wastes consisting of finely divided insoluble matter such as is produced by cellulose, paper, cloth, cotton and rag mills, and also by mines and iron works, are all injurious by their tendency to lodge in the gills. Such waste matter may be either flocculent, or in the form of fine hairs or fibers, or sharp-edged hard grains. My own observations indicate in general that such is not quite the case. A healthy fish has the power to eject all such foreign bodies from its gills. If, however, the gills are irritated by only small quantities of acid, alkali, or even salts, these organs endeavor to protect themselves against the action of such chemicals by exuding a considerable quantity of mucus upon their surface; and the

[1] It was asserted by von Pettenkofer (*Archiv f. Hygiene*, vol xii, page 269) that a quantity of municipal sewage might be allowed to enter a stream up to one-fifteenth of the minimum stream-flow, without causing any objectionable conditions, provided the current in the stream was not lower than the velocity of the sewage in the sewers.

damage is then done by the adhesion of the particles of cellulose, textile fiber, or other matter to this mucus and the accumulation of such matter thereon, until the gills become so completely covered as to cause suffocation. A copious secretion of mucus by the gills is induced by very small amounts of such chemicals in the water. When the chemicals are present in large quantity or strongly concentrated, the gills are destroyed directly by disintegration.

"If acids are discharged into a stream, a new factor for the self-purification of the water is introduced by the neutralizing capacity of the dissolved bases. Thus the bicarbonate of lime usually carried in solution in the water will neutralize a certain quantity of the acids; and in the case of sulphuric acid a combination with lime will occur, whereby the resulting insoluble sulphate of lime or gypsum will settle on the bottom. Such a self-purifying process, however, is not a natural one, as the acid of the sulphate in the mud is sometimes reduced by other processes to sulphuretted hydrogen. This latter is a well-known poison for fish, but it may be rendered harmless by the operations of the sulphur bacteria, such as the beggiatoa previously mentioned.

"Similarly, if caustic lime is thrown into the stream, it will ultimately be rendered harmless to fish by combining with the free carbonic acid in the water and forming either insoluble carbonate or soluble bicarbonate of lime, the latter being a normal component of many natural waters. This process, however, is not very rapid, and before its completion the water must generally flow through long distances."

BALANCED AQUARIUM

In brief, it may be said that the plankton forms an essential element in the food cycle of aquatic life, and that under normal conditions there is an exact balancing of the various forces which include the plankton and bacteria, the larger fish life, the amount of dissolved oxygen and other chemical constituents of the water. This is illustrated by the "balanced aquarium," thus described by Dr. George W. Field:

"The 'balanced aquarium' is a familiar example on a small scale of what is constantly taking place in nature. In such an aquarium the plants assimilate the nitrogenous material excreted by the fish and other animals, after this nitrogenous material has been oxidized, in whole or in part, by bacterial action. The plants, also, use as food the carbon dioxide coming mainly from the animal respiration. In return, the plants (diatoms and algæ) give out oxygen which supplements that absorbed from the atmosphere at the surface, and thus aid to a most important extent in maintaining a continuous supply of free oxygen in the water; for, though they require a certain amount of oxygen, they liberate into the water much more oxygen than they consume.

"In the days when natural conditions obtained, previous to the pollution of the waters of Stony Brook and of Muddy River (near Boston, Mass.), in the region of the present basin, the total number of plants and animals per

cubic centimeter was undoubtedly greater than at present. It is safe to say that the plants and animals were so adjusted to their environment that offensive conditions due to the death, decomposition or putrefaction of organic matter did not exist; the conditions represented essentially a balanced aquarium on a large scale.

"The surrounding water then contained an abundance of oxygen, and the circulation of water in the large original area went on under the influence of currents and winds. With the increase of population, abnormal conditions gradually became predominant, chiefly the increasing introduction of crude sewage and the progressive encroachment upon the area of the basin by filling; until today the immensely increased amount of nitrogenous matter, poured into the much-restricted basin, whose diminished area is less favorable for deriving oxygen from the air, leads to a very rapid exhaustion of the oxygen in the water, followed by putrefaction and the increase and accumulation of the products of putrefaction. This results in conditions fatal to those plants which are unable to withstand or adapt themselves to the changed conditions of life. This loss of the oxygen-producing plants reduces the capacity of the water to oxidize dead organic products, since the water is now almost entirely dependent upon the air for its supply of oxygen, instead of being able to use the supplementary supply furnished ordinarily by the diatoms and green algæ. Its capacity for absorbing sewage is thereby still further diminished. We may say that, other things being equal, the water which contains a large quantity of diatoms and green algæ can dispose of more sewage than water which contains less of these plants or none at all." (Report on Charles River Dam, page 328.)

The interdependence of the various factors is thus strikingly set forth by Dr. Edward A. Birge and Dr. Chancey Juday in Bulletin 22 of the Wisconsin Geological and Natural History Survey:

"The inhabitants of an inland lake form a closed community in a stricter sense, perhaps, than the term can be applied to any other non-parasitic assemblage. The number of species living under these conditions is small and closely similar in different lakes. Only small additions are made to the food supply from without and these come slowly. The lake is dependent on its own stock of green plants for the stock of organic matter available for food of other organisms; and the possible amount of green plants is limited by the raw material supplied for photosynthesis from the lake itself (page 15).

"Our experience has led us to expect that certain individual peculiarities will recur in certain lakes. These closed assemblages of plants and animals have lived together, in a limited environment, since a time which goes back nearly to the glacial period. It seems as if they had acquired certain habitual actions and reactions upon each other and on the environment, whose details differ in individual cases—physiological variations in the assemblage and its units, not unlike the morphological variations seen in the forms of Daphne from different lakes" (page 16).

EFFECT OF BRACKISH WATER UPON PLANKTON

In the case of tidal waters varying salinity is one of the causes tending to disturb the balance mentioned above. In this matter osmosis undoubtedly plays an important part.

"This (osmosis) is the tendency of solutions of different degree to mingle by passage of water through a permeable membrane. The membrane is permeable to water molecules, but not to the dissolved salt molecules. The osmotic pressure of sea water is 0.65 atmosphere for each 1 per cent. of salt content, or a head of more than 6 m. This is of importance biologically. Organisms that live in water generally have a skin that is not impermeable to water, but they are also adapted to a certain degree of salinity of the water; hence if they are suddenly put in water of lower salinity, the water will pass through the skin into the body, and conversely, they will exude water. Both of these actions are usually unfavorable to the life of the organism, and, therefore, the organisms endeavor to avoid such conditions as much as possible. Osmotic pressure thus tends to confine most marine animals within certain strata, and frequently the stratum is thin, corresponding to a narrow range of salinity." (Kayser, "Physik des Meeres," 1911, translated by Kuichling.)

If this theory is true it follows that the plankton of fresh water cannot thrive in salt and brackish water, and marine plankton cannot thrive in fresh or brackish water. Hence a body of salt water into which a fresh-water stream flows, being subject to variations in salinity due to the relative proportions of fresh and salt water, is not well adapted to the plankton, and it may be expected that the normal plankton of fresh water and the normal plankton of salt water will be present in diminished numbers and diminished also in activity, or that species of plankton will be found which have the power of adapting themselves to the variable conditions. There are some reasons for believing that such bodies of water have not as great a capacity for disposing of sewage as other bodies of fresh or salt water. Very little, however, is known upon this subject, and a positive statement of the relative capacities of waters of varying salinity cannot be safely given at this time.

"Diffusion is a similar action to osmosis; in fact, it is identical therewith. Diffusion is the tendency of solutions of different degree of salinity to mingle. The dissolved salt has the same tendency in water to occupy the greatest possible space as a gas in a vacuum. This action takes place both in pure water of indefinite volume, and when the solution is separated from the pure water by a membrane permeable to the water." (Kuichling's translation of Kayser's "Physik des Meeres.")

An actual example of the effects of varying salinity is cited by Field (Report on Charles River Dam, page 338). He made a close study of Point Judith Pond which was subject to a variable mixture of fresh and

salt water on account of frequent breaches made by the sea through a narrow neck of land separating the pond from the ocean. As a result of these studies a permanent opening to the sea was made, resulting in a constant degree of salinity and in improved conditions summarized thus:

"1. The condition of varying salinity arising from a complete dam and a basin into which fresh water was entering constantly, with salt water introduced at irregular intervals, was harmful.

"2. The differing specific gravity (fresh water on top and brackish or salt water below) was accompanied by imperfect oxygenation of the water.

"3. The varying salinity and resultant plasmolysis, together with defective oxygenation, were followed by death and putrefaction of organisms in the basin.

"4. Restoring a more constant salinity, with its more constant specific gravity, removed the offensive conditions and renewed the growth of organisms."

BIOLOGICAL CONDITIONS IN GENESEE RIVER AND LAKE ONTARIO

A study of the biological conditions in the Genesee River was made in 1912 by Whipple in the course of an investigation of the effect of the sewage of Rochester on that river and Lake Ontario. He found a very great variation in the numbers of bacteria in the river on different days. On July 15, for instance, about 500,000 per cc. were found near a sewer outlet; this number increased downstream to 2,000,000 and then decreased to only 9000 at the river mouth. On July 30, there were 750,000 bacteria at the surface of the river at its mouth and 90,000 at the bottom. The general deduction by Whipple from his examinations was that a very large proportion of the sewage bacteria failed to reach the lake during the period covered by the observations, being eliminated by processes of self-purification in the river.

An important factor in the destruction of the bacteria was stated in Whipple's report to be the action of the larger microscopical organisms. Immediately below the outlet of the trunk sewer was a zone of heavy pollution, within which the bacteria were very numerous and the crustacea relatively few. From this point downstream, the numbers of bacteria gradually decreased. In the vicinity of the intense bacterial pollution and for a short distance below it, the numbers of protozoa were high, as might be expected from the fact that these organisms consume bacteria. At the same time Whipple found a slight increase in the algæ, and their numbers were well maintained to the river mouth; these vegetable organisms, it has been pointed out, consume the oxidized products of the organic matter from the sewage. In the lower course of the river, for 2 or 3 miles back from the mouth, the rotifera and crustacea

made their appearance; these live on the algæ and bacteria and particularly on the protozoa. Large numbers of crustacea were found in the lake around the mouth of the river. Table 15, compiled from two tables in Whipple's report, gives an approximate idea of these changes.

TABLE 15.—APPROXIMATE DISTRIBUTION OF MICRO-ORGANISMS IN GENESEE RIVER AND LAKE ONTARIO

Numbers per Cubic Centimeter (Whipple)

Place of sampling	Miles from mouth	Bacteria		Algæ		Protozoa		Crustacea	
		Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
River.....	8	15,000	700	580	0	48	0	100[1]	0
River.....	6½	430,000	31,000	520	120	156	8	100	0
River.....	6	750,000	108,000	504	206	2,536	0	100	80
River.....	5½	2,040,000	350,000	197	93	356	94	150	0
River.....	4	2,740,000	713,000	441	161	2,836	979	100	33[1]
River.....	2	340,000	8,000	538	293	501	0	475[1]	53
River.....	½	247,000	Unknown	443	250	330	23	850	50
River.....	0	190,000	4,000	616	252	110	10	550[1]	48
Lake	¼	....	....	......	......	......	......	1,168[1]	......
Lake......	½	....	....	......	......	......	......	1,175[1]	......
Lake......	1	....	....	......	......	......	......	55[1]	......

[1] Including rotifera.

EFFECT OF EXTENT OF POLLUTION

The greatest influence on plankton in its relation to sewage disposal is the quantity of nitrogenous organic matter in the water. While a certain quantity of nitrogen is necessary for the growth of plankton, too much of it in the form of complex organic matter of sewage causes the development of bacteria, depletes the oxygen in the water, and leads to the death and decay of the microscopic organisms. For, although much of the plankton liberates oxygen to the water, it is necessary on the other hand for it to take up some of this dissolved oxygen in the water, in order to live. Field says:

"In regard to the effect of sewage pollution upon the flora and fauna of water in general there are, so far as I know, few precise data on record. It is well known, however, that in Narragansett Bay (Rhode Island), the progressive increase of pollution has driven farther and farther down from the apex of the bay at Providence the zone within which such forms of plant life as diatoms and algæ prevail, and has replaced this former wealth of life by lower forms, chiefly putrefactive bacteria, causing conditions not very dissimilar to those now prevailing in the Fens basin (Charles River Basin in Boston, Mass.). No bacterial counts or chemical analyses are available which show the degree of dilution or the average chemical constitution at

the edges of these zones beyond which the diatoms, green algæ and animal life find the conditions of life too difficult for them to thrive.

"This line in a general way marks the border beyond which the conditions become permanently offensive, and about all that can be said, without further research, is that such a line exists. That this delicate line, beyond which the proportion of nitrogenous matter becomes injurious to life and growth, is rather sharply defined, was indicated by a series of experiments by Vernon, at the Naples Zoological Station, made with a view to improving the quality of water in the large aquaria (the prototype of those at the aquarium, Castle Garden, New York), which showed that the growth of young echinoderms (Plutei) was increased by the addition of 30 parts per 100,000 of potassium nitrite, or 105 parts per 100,000 of potassium nitrate, while larger proportions checked growth; and that the addition of 4 parts per 100,000 of ammonium chloride greatly increased the mortality.

"It will be noted that these amounts which could be taken before injurious effects appeared were very large; but the products of organic decomposition, of which little is known, are probably of greater influence on life than these inorganic salts." (Report on Charles River Dam, page 326.)

Birge and Juday, of the Wisconsin Geological and Natural History Survey, made a very thorough study of the amounts of dissolved gases, together with their relation to the plankton, in the lakes of Wisconsin, and have set forth their data and conclusions in great detail in Bulletin 22 of the Survey. The plankton with relation to sewage disposal has been studied but slightly in America, and although it is generally known on theoretical grounds that polluted waters should support a vigorous growth of plankton, many factors as yet unknown are involved, which mask the results and make it impossible to state definitely now the influence of pollution in any particular instance. Birge and Juday say:

"One general question may be named, whose answer is as yet almost wholly outside our knowledge and yet is of great importance in every way. Why is it that different lakes differ so widely in productivity or in ability to support a population of plankton? The lakes are equally old; they may have the same species of plankton; their temperatures may not differ widely; the chemistry of their water is not greatly different; they have had apparently the same chance for development; yet the results are very unlike. This is a problem whose solution will demand the answer of many subsidiary questions. These will concern not only the biology of the several plankton species but the study of the relation of the collective plankton to the lake and of the interrelations of its members. Such matters, for instance, as the food supply of the algæ in the lakes as influenced by the inorganic and organic substances dissolved in the water, by the area and depth of the lake, and the form of the lake basin; the effect of the several crops of algæ on their successors, by withdrawing certain substances from the zone of photosynthesis and adding others—these are examples of questions whose solution demands not merely a knowledge of the biology of the several species of algæ,

but also the study of the several lakes as physiological individuals of a higher order." (Bulletin 22, Wis. Geol. and Nat. His. Survey, page 19.)

Whipple says in his "Microscopy of Drinking Water:"

"In spite of the vast amount of study that has been given to the microscopic organisms, we are still very far from understanding the laws governing their distribution. Why it is that a certain genus will grow vigorously in one pond and at the same time be absent from a neighboring one where the conditions apparently are as favorable, or why a form may suddenly appear in a pond where it has been never before seen, we are still unable to say with certainty. Solution of such problems involves a far-reaching knowledge of the chemical constituents and the life-history of the organisms, besides the effect of physical conditions, such as temperature, pressure, light, etc. The sciences of bio-chemistry and bio-physics are yet in their infancy. Until these have been further developed many problems connected with the microscopic organisms must remain unsolved" (page 137).

"An important question, and one which is of particular interest to water analysts, is the relation between the growths of organisms and the chemical analysis of the water in which the organisms are found. Unquestionably there must be such a relation, but thus far our knowledge of the food requirements of the plankton is not sufficient to enable us to tell what this relation is" (page 142).

He also summarizes the results of monthly examinations of 57 ponds and reservoirs in Massachusetts together with the chemical and physical characteristics of the same waters. From these comparisons he finds that an excess of chlorine, meaning the difference between the amount found in a sample and that found in a normal or uncontaminated water of the same region, influences only to a slight degree the quantity of plankton. This fact corresponds with the common observation that vigorous growths of organisms are often observed in ponds far removed from any possible contamination. He further states:

"Since nitrogen is essential to all living matter we naturally expect that organisms will thrive best in waters rich in that element. The above statistics show that this is the case, and that it is true for each class of organisms and for the different conditions of nitrogen tabulated. The free ammonia and nitrates appear to be particularly influential in determining the amount of life present. For example, 10 of the 13 ponds low in free ammonia never show maximum growths of the cyanophyceæ above 100 per cubic centimeter while 4 of the 7 ponds high in free ammonia commonly have growths above 1000 per cubic centimeter.

"One must be careful in these matters, however, not to mistake cause for effect. Free ammonia, for example, indicates organic matter in a state of decay, and instead of representing the food of the organisms in question it may represent their decomposition. The interaction of the various organisms is a very complicated question, and the extent to which one organism lives upon the products of decay of another is not well known" (page 146).

BIOLOGY OF SEWER OUTFALLS

In Germany, studies of the biology of sewer outfalls have been made by Kolkwitz, ("Biologie des Trinkwassers, Abwassers und der Vorfluter," 1911). He divides the water into three zones: (*a*) the sewage zone with polysaprobic organisms, (*b*) the transition zone with mesosaprobic organisms, and (*c*) the clean water zone with oligosaprobic organisms. Kuichling's translation gives the following statement regarding the zones:

"**Polysaprobic Zone.**—This zone is characterized by the multitude of Schizomycetes, bacteria-eating Fagellata and Ciliata . . . From a chemical standpoint, this zone is also marked by the predominance of reducing and splitting processes, due to lack or insufficient supply of dissolved oxygen and to the excess of carbonic acid; also to its relatively high content of multimolecular, nitrogenous and decomposable food matters, such as peptides and amino-acids, which is indicated by the growth of certain organisms that may be regarded as living reagents.

"**Mesosaprobic Zone.**—In the mesosaprobic zone of a river, the process of dissociation or decomposition has advanced to an intermediate degree. For convenience this zone will be divided into 2 parts, called A-mesosaprobic and B-mesosaprobic, the first extending upstream and adjoining the polysaprobic zone, and the second extending downstream and adjoining the clean water or oligosaprobic zone. The first part is usually characterized by the appearance of Schizophyceæ and Eumycetes, along with Anthophysa vegetans, Stentor cœruleus, Carchesium lachmanni, etc. The water still contains many bacterial germs, reaching in number hundreds of thousands. Drainage ditches carrying imperfectly purified water are good examples of A-mesosaprobic zones. The second part, or B-mesosaprobic zone, is generally distinguished by its contents of Diatomaceæ, Baccillariaceæ and certain Chlorophyceæ, along with Rhizopoda, certain Ciliata, Vermes, numerous Rotatoria, etc.

"**Oligosaprobic Zone.**—The hygienic interest in the disposal of sewage generally terminates at this zone, while it begins therewith in the case of domestic water supplies taken from surface-water sources. The chief characteristic of this zone is the completion of the process of mineralization; all of the more or less turbulent processes of self-purification are absent in the water itself, and the biological development is usually extensive. The appearance of certain representatives of the Peridiniales, all Charales, and certain planktonal Ciliata, Rotatoria and Crustacea, are to be regarded as characteristic."

"**Summary.**—In summarizing the various factors that are effective in the biological self-purification of water the following points will be recognized:

"1. Splitting and reduction of organic substances by bacteria.

"2. Oxidation by transpiration and allied processes.

"3. Removal of dissolved organic food matters by fungi and algæ, and the transformation thereof into living matter.

"4. Consumption of organic solid fragments by animals, or the transformation of such matter into living substance by detritus and carrion eaters.

"The organisms participating in these four processes are exciters of putrefaction and also reducers thereof. They finally accomplish mineralization by removing the dead substances.

"5. Destruction of aggregations of bacteria (including pathogenic ones) and algæ by various bacteria-eaters.

"6. Destruction of small crustacea, etc., by fishes.

"The two processes indicated in 5 and 6 govern the circulation of living matters. Although millions of minute crustacea may be necessary to supply the food for young fishes, thousands of millions of microscopically small organisms are required for the nutrition of the crustacea.

"7. Production of oxygen by chlorophyllaceous organisms, which serve to aerate the water and remove carbonic acid.

"8. Aeration of decomposable sediments by animals that burrow in the mud and sludge.

"The two processes indicated in 7 and 8 govern or accelerate self-purification.

"The most ideal purification of the water would be accomplished by the gasification of the matters contained therein. This takes place now to considerable extent, as the following gases escape from the water into the atmosphere: CO_2, NH_3, H_2S, N, H, and CH_4. Many winged insects also emerge from their aquatic larvæ."

PLANKTON IN STREAM RECEIVING EFFLUENT FROM INTERMITTENT SAND FILTERS

A residential city of about 15,000 population in Massachusetts treats its sewage in sedimentation basins and on numerous intermittent filter beds of fine sand thoroughly underdrained. The sewage is strong, about 3 hours old when it reaches the treatment plant, and averages 450,000 gal. or about 40 gal. daily per capita served. The filter effluents are discharged into a brook having a drainage area of about 0.92 square mile about $\frac{1}{4}$ mile below the beds, of 1.83 square miles at the first dam on the brook, and about 2.81 square miles at a second dam. The normal yields of this brook for the months of July and March are given in Table 16.

TABLE 16.—MINIMUM AND MAXIMUM NORMAL MONTHLY YIELD OF BROOK BELOW SAND FILTERS IN A MASSACHUSETTS CITY

Month	At highway $\frac{3}{8}$ mile below filters		At first dam $\frac{5}{8}$ mile below filters		At second dam $1\frac{1}{8}$ miles below filters	
	Gal. per day	Ratio of sewage to stream flow	Gal. per day	Ratio of sewage to stream flow	Gal. per day	Ratio of sewage to stream flow
July....	178,000	1:0 395	353,000	1:0.785	542,000	1:1.205
March...	2,681,000	1:5 96	5,332,000	1:11.85	8,190,000	1:18.20

Note.—Sewage flow assumed to be same in March as in July; about 40 gal. per day per person served by sewers, or 450,000 gal. per day.

TABLE 17.—CHEMICAL ANALYSES OF WATER FROM BROOK AND PONDS CONTAINING PLANKTON LISTED

(Parts per million and numbers per cubic centimeter)

	Effluent from filter beds	Outlet from first pond	Outlet from second pond
Oxygen consumed	16.40	15.90	16.90
Nitrogen as:			
Free ammonia	2.2	1.7	2.7
Total albuminoid ammonia	0.2	1.28	2.32
Suspended albuminoid ammonia	2.0	0.424	0.38
Dissolved albuminoid ammonia	2.0	0.60	0.50
Nitrites	13.8	5.4	3.6
Nitrates	12.0	8.0	7.0
Chlorine	9.75	7.75	7.50
Diatomaceæ:			
Cyclotella	....	100	100
Navicula	....	300	100
Synedra	....	1,600	1,800
Amphora	....	100	....
Diatoma	....	400	....
Chlorophyceæ:			
Pediastrum	....	1,600	100
Protococcus	20	27,300	25,900
Scenedesmus	....	3,000	3,900
Raphidium	....	1,100	1,100
Cyanophyceæ:			
Chroöcoccus	....	....	200
Microcystis	....	200	....
Fungi and Schizomycetes:			
Crenothrix	20	....	....
Mold hyphæ	110	....	....
Protozoa:			
Peridinium	10	....	100
Synura	....	....	100
Trachelomonas	....	300	....
Monas	10	....	....
Colpidium	10	....	....
Chlamydomonas	....	1,200	1,200
Total	180	37,200	34,600

The brook has a current of 1 to 2 ft. per second for a distance of about ¼ of a mile to the first pond, which has a capacity equivalent to from 20 to 30 days' discharge of the brook during the dry season, including the effluent of the sewage filter beds. From this pond the water flows in a sluggish stream ⅛ of a mile to the second pond, which has a capacity approximately the same as the first.

At times when the effluent from the filters has been only moderately high (below 10 parts per 1,000,000) in nitrates, leptomitus appeared in large quantities upon the bed of the effluent brook. This fungus, however, almost entirely disappeared by 1913, when the effluent became extremely high in nitrates and the quantity of organic matter low.

The growth of plankton in the ponds below the filter beds is remarkable during the summer seasons. The plants are present in such quantities that they give to the water and to the pond when viewed from a distance a decidedly greenish color. During August, September and October, temperature and other conditions appear to be much more favorable to the growth of the plankton than during other seasons. Below the second pond, the plankton gradually diminishes in numbers, apparently because its food supply has been too greatly reduced to support it in the large numbers found in the ponds above.

The results of chemical analyses and microscopic examinations by Prof. Robert Spurr Weston of the water in the effluent brook and in the first two ponds, are given in Table 17. The results of the chemical analyses indicate changes going on in the water which must be attributed largely to the micro-organisms. It would appear that the organisms had laid hold of the organic matter dissolved in the effluent and transformed it into their living bodies, in which form it appeared in the water at the outlet to the lower pond as suspended organic matter. The chlorine indicates that the flow from the second pond was made up of 3 parts of effluent to 1 part of natural stream flow. The important effect of the micro-organisms in maintaining the supply of oxygen in this stream is indicated by analyses made when the effluent was relatively high in free ammonia and low in nitrates. At this time the dissolved oxygen decreased as the water flowed along the brook until it reached the first pond, in which the oxygen content was greatly increased, due to the life processes of the organisms. The results of these analyses were as follows:

Place of sampling	Saturation of oxygen, percentage
Brook just below filter beds	41
Brook ¼ mile below beds	18
First pond near outlet of brook	169
Outlet of first pond	168

EFFECT OF ALGÆ IN MUDDY CREEK, CINCINNATI

Muddy Creek is a small stream nearly dry during the summer season, except after storms, having a drainage area of about 17 square miles and flowing into the Ohio River just below Cincinnati. Upon the upper portion of its drainage area is the town of Westwood with an area of perhaps 1½ square miles and a population of about 1700 persons. This community has a system of sewers with which most of the houses have been connected. The sewage is discharged into the upper reaches of Muddy Creek and flows first in a sluggish stream for a distance of about 1 mile, after which the flow is spread out in a thin stream and in pools upon the wide and rocky bed of the creek.

At times odors arising from the upper end of the creek immediately below the mouth of the sewer have given rise to serious complaints and the water in this part of the creek is apparently at times low in dissolved oxygen. On Nov. 9, 1912, an examination made under the direction of the authors showed that the creek was dirty in appearance at its upper end, as would be expected from the discharge of sewage into a practically dry brook. Some ground water was mixed with the sewage and the water, even at the upper end, contained about 70 per cent. of its saturation value of dissolved oxygen. As the water flowed along the creek, however, a green growth of algæ developed to such an extent that enormous numbers of gas bubbles could be seen breaking away from the masses of green. By holding a sample bottle over the bed of algæ and moving it gradually along, a sample of this gas was collected which, when tested by being ignited, appeared to be pure oxygen. At a distance of 1¾ miles below the outlet of the sewer, the water was 130 per cent. saturated and free from objectionable appearance, a marked illustration of the purifying capabilities of the organisms living in it.

NITROGEN AND CARBON CONTENT OF MICRO-ORGANISMS

Comparatively little investigation of the chemical composition of micro-organisms has been made. The blue-green algæ are very high in nitrogen, anabena has over 7 per cent. of nitrogen, and oscillaria has as high as 11 per cent. The fungi have from 4 to 8 per cent. of nitrogen and the green algæ have percentages of nitrogen lying between the fungi and the blue-green algæ. In general the quantity of nitrogen found in the different classes of organisms lies between the following quantities:

	Per cent.
Fungi	4.0–8.0
Green algæ	6.0–9.0
Blue-green algæ	6.5–11.5

Taking saccharomyces as typical of fungi and oscillaria of the blue-green algæ, the carbon, nitrogen and ratio of nitrogen to carbon, which may be expected in organisms of these classes are as follows:

Organisms	Carbon, per cent.	Nitrogen, per cent.	Ratio, nitrogen to carbon
Saccharomyces	49.0	7.1	1:6.9
Oscillaria	34.2	7.9	1:4.3

LEMNA OR DUCKWEED

According to some classifications lemna can be included in plankton only by courtesy although other definitions of plankton would include it. In certain parts of the country duckweed is frequently found in ponds and lakes high in nitrogen, although free from sewage contamination. In fact, lemna has been found living in municipal water works reservoirs when the water was perfectly wholesome and safe for consumption.

An analysis of duckweed taken from the first pond below the sewage filters previously mentioned, made by Prof. Joseph H. Perry of Worcester, furnished the results given in Table 18.

TABLE 18.—CHEMICAL ANALYSIS OF LEMNA OR DUCKWEED

	Proportion of organic and inorganic matter, per cent.	Proportion of organic matter, loss on ignition, per cent.	Ratio, carbon to nitrogen
Ash	15.14		
Carbon	41.53	48.94	11.30
Nitrogen	3.68	4.33	
Hydrogen	5.21	6.25	
Oxygen (by difference)	34.44	40.48	

CONCLUSION

While in recent years considerable study has been given to the plankton and the conditions under which it may be found and may thrive, little can be said with assurance of accuracy of the amount and character of work which it accomplishes. This is a field of research which gives promise of important results if only the fundamental principles can be discovered upon which the development and control of the organisms depend. That their mission is to convert substances like those in sewage and effluents into living tissue is clear, but how to take advantage of this action, to accomplish the most good under the many conditions encountered, is not evident from present knowledge.

CHAPTER V

COMPOSITION OF SEWAGE

The composition of sewage varies widely. In newly constructed sewerage systems, before connections have been made, there is often a considerable flow consisting wholly of ground water. Years ago, when water-closet or cesspool drainage was not allowed to enter the sewers the liquid flowing in them was very different in composition from modern sewage.

Definitions of Sewage.—Sewage was defined by Prof. Mansfield Merriman as "water fouled with soap, vegetable and animal matter, urine and feces" ("Elements of Sanitary Engineering," page 139); by Kinnicutt, Winslow and Pratt as "the water supply of a town or city after it has been used" ("Sewage Disposal," page 1); by George W. Fuller as "the spent water supply of a community . . . supplemented in some instances by street washings and industrial wastes." ("Sewage Disposal," page 1.)

From consideration of the sources of sewage it seems logical to define it as a combination of the liquid wastes conducted away from residences, public and business buildings and industrial establishments, in pipes, conduits or open ditches, including such ground water as may reach these conduits, together with storm water, if the sewers are built in accordance with the combined plan, and excluding storm water if on the separate plan.

Kinds of Wastes in Sewage.—The liquid wastes from kitchen sinks are called "sink drainage" and the conduits through which they flow "sink drains." When to these are added the wastes from laundry tubs, bath tubs and water-closets, the combined wastes are properly termed "house sewage," for they have been collected by a part of the building's plumbing system and conveyed from the house by the house sewer or house drain (Volume I, page 38). The wastes from a manufacturing process are generally termed "industrial," "manufacturing," "manufactural" or "trade" wastes. Often sinks and dry or water-closets used by employees discharge into the pipes carrying trade wastes, and there is then some question whether the combined liquids may be termed sewage. This is largely a question of degree, and the answer depends principally on the relative proportion of manufacturing wastes and house sewage.

Rain water running directly from roofs is called "storm water" or

"roof water." That running off the ground is called "storm water," and that running from streets is sometimes termed "street wash," because it is the water with which the street surfaces have been washed. All storm-water run-off is wash water, and has washed the surfaces over which it has flowed, frequently taking up large amounts of impurities in doing so. The term "storm water" should be applied only to that portion of the rainfall which runs off during or shortly after a storm. Ordinary stream flow is distinguished from storm water by designating it as "surface water." This term includes storm water, but not those waters which are retained in or flow through the ground, the latter being called "ground waters." Waters escaping from the ground into streams and lakes become "surface waters."

Origin of Sewage.—Starting with the simplest sewerage community, a small group of houses drained by a single sewer, we have from each of the houses the "house sewage" flowing from the "house sewer" to the "sewer." At this point it may be said to have changed from "house sewage" to "sewage." No individual house sewage changes its composition by its change of location, but it mingles with other house sewages coming from buildings where the activities were different and consequently where the house sewage was different. In some houses the waste water from the early morning baths was being discharged; in others, sink water from the breakfast dishes; from still others, perhaps, the laundry wastes. The sewage from this simplest sewerage community thus consists of mingled house sewages of different qualities, but still remains "domestic sewage."

Passing on down the sewer the sewage of the simple sewerage community is mingled with other domestic sewages from other simple communities and with them the wastes from business houses, hotels, stables and garages. If there are industries like dye works, breweries or woolen mills, the spent dyes, vat and barrel washings, and cloth wash water, may be discharged into the sewer, materially modifying the quality of the sewage. In many places ground water finds its way into the sewers, sometimes in very considerable quantity, having a marked effect on the quality of the resulting sewage.

Storm water is not intentionally admitted to separate sewerage systems, but in many cases there may be leakage through manhole covers, some street inlets may be connected with the sewers and roofs may be connected, either with or without permission from the authorities. Such storm water may also have a marked influence upon the quality of the sewage in diluting it. The authors have known a dry-weather flow of 300,000 gal. per day, in a separate system of sewers, to be increased to 3,000,000 gal. by storm water and ground water finding their way into the sewers.

Combined sewers are designed to take storm water, for which inlets

are provided on the streets, and in such sewers the composition of the sewage is materially influenced at times by storm water. In the early part of a storm large quantities of mineral and organic matter are contributed by the street washings, and the roof water and run-off from other areas, reaching the sewers somewhat later, afford considerable dilution. Sometimes there are considerable deposits of sewage matter in old sewers which are washed along by the storm water. Such deposits are often extremely foul and high in organic matter. The effect of this material and the wash of the streets is often to make the early part of the storm flow much stronger than the ordinary sewage. Such flows are frequently alluded to as "first flushings."

SUBSTANCES PRESENT IN SEWAGE

In outlining the object of the work of the Lawrence Experiment Station of the Massachusetts State Board of Health in 1890, Hiram F. Mills made the following statements:

"The object is to learn how to purify sewage. Sewage varies much in the amount of impurity it carries, depending upon the amount of water used. It is much more dilute in American than in European cities. Here a sewage stronger than ordinary would contain, say, 998 parts of pure water, 1 part of mineral matter and 1 part of animal and vegetable matter. The animal and vegetable matter are, for convenience, classed together and called organic matter. Sewage would become entirely purified if we should take out the 2 parts of mineral and organic matter and leave the 998 parts of pure water; but, as the mineral matter is not generally objectionable, we are satisfied to call it purified if we succeed in taking out the 1 part of organic matter. Of the 2 parts of mineral and organic matter in 1000 parts of sewage, about one-half is in suspension, and can be strained out by the finest strainer that water will pass through; the other half is dissolved in the water, and cannot be thus strained out. If a way can be devised by which the impurity of sewage can be completely oxidized or burned, so that inorganic or mineral matter only remains in the otherwise pure water, the result will evidently be satisfactory. To learn how to do this is the main object of our work." (Report Mass. St. Bd. Health, 1890, Part II, page 5.)

While the quantities of mineral and organic matter suggested by Mills, (2000 p.p.m.[1]), are much larger than an average American sewage contains, this description nevertheless gives a general idea of the very small quantity of such matter present in proportion to the quantity of water. The authors have often referred to the organic and mineral matter as constituting about 0.1 per cent. of sewage.

It is a popular opinion that sewage is the municipal water supply defiled by use and discharged into the sewers. Were this wholly true

[1] In this chapter p.p.m. is used to signify parts per million wherever that term follows numbers.

the quantity of sewage would correspond closely with that of the water supply. In a general way it does, but the whole of the water supply does not reach the sewers (Volume I, page 166) and much water from other sources enters them, such as ground water and private water supplies. It is, therefore, important to consider as the basis of sewage the composition of the water supplies and ground water, and to estimate the quantity of each entering the sewers.

TABLE 19.—COMPARATIVE ANALYSES OF WATER AND SEWAGE
(Parts per million)

Place	Residue on evaporation	Nitrogen as			Chlorine	Nitrogen as		Oxygen consumed	Hardness
		Free ammonia	Total albuminoid ammonia	Suspended albuminoid ammonia		Nitrates	Nitrites		
Brockton, Mass.									
Water supply (1905-09)	32.7	0.013	0.103	0.019	6.2	0.009	0.000	2.2	5.0
Sewage (1905-09)......	2210.0	47.8	24.5	19.3	144.3	.....	.....	433.7	......
Worcester, Mass.									
Water supply (1905-09)	31.4	0.019	0.120	0.021	2.1	0.052	0.001	3.0	8.7
Sewage (1905-09)......	871.0	17.7	7.2	4.3	113.4	.....	.....	122.0	......
Providence, R. I.									
Water supply, filtered (1910).	56.0	0.009	0.078	0.002	6.3	0.12	0.00	3.5	14.0
Sewage (1909).........	1715.0[1]	15.40	7.00	3.50	496.5	.....	.....	89.3	......
Chicago, Ill.									
Water supply (1908)..	156.0	0.034	0.104	......	5.0	0.29	0.00	2.6	......
Sewage (1909-10)......	471.0	8.8	7.6[2]	......	40.0	0.35	0.11	38±	......
Mansfield, Ohio.									
Water................	391.0	0.081	0.012	......	8.5	1.5	0.0	0.05	285.0[3]
Sewage...............	876.0	13.3	31.7[2]	......	108.7	0.2	0.0	51.6	......

Brockton sewage from Brockton city reports. Worcester sewage from Worcester city reports. Brockton and Worcester water analyses from Mass. State Bd. Health Rept., 1909, page 198. Chicago, Langdon Pearse and Report of Lake Michigan Water Commission, 1909, page 45. Providence water analysis, Pratt. Providence sewage analysis, Bugbee. Mansfield, Ohio, Dittoe.

[1] Estimated.

[2] Organic nitrogen (about twice the nitrogen as albuminoid ammonia).

[3] Average of Hedges spring supply and supply from main pumping station wells. Alkalinity plus incrustants.

The average analyses of a few water supplies and the corresponding sewages are given in Table 19. The water supplies and sewages of Brockton and Mansfield illustrate the point just made. If the water supply and ground water at Brockton had contained the same amount of solids as those at Mansfield, and the same amounts as at present were contributed by the wastes, then about 17 per cent. of the total solids in the sewage would have been furnished by these waters. In other words,

waters which are high in residue, (which is the case with all very hard waters), contribute a considerable proportion of the solids in the corresponding sewage, while waters low in residue furnish to the sewage only an insignificant proportion of its solids.

The quantity of nitrogenous matter in municipal water supplies is usually so low, see Table 19, that its influence upon the composition of the sewage may be disregarded. At Worcester, Table 20, the organic solids in the water, 10.5 p.p.m., are equivalent to about 1.19 per cent. of the total solids and 2.45 per cent. of the organic solids of the sewage.

TABLE 20.—SOLIDS IN WATER SUPPLY AND SEWAGE AT WORCESTER, MASS. 1910

(Quantity of sewage = 107 gal. per capita per day. Water consumption 72 gal. per capita per day)

	Water supply		Sewage	
	p.p.m.	Grams per capita per day	p.p.m.	Grams per capita per day
Total solids	31.4	12.7	882	365
Organic solids[1]	10.5	4.7	429	178

[1] Estimated.

To the substances contained in the water are added urine, feces and paper from the water-closets. Researches of Wolff and Lehmann (First Report, Rivers Pollution Commission of 1868, page 27), gave the quantities of urine and feces and their constituents as stated in Tables 21 and 22. ("Sewage Disposal in the United States," Rafter and Baker, 1894, pages 155–157.)

The Metropolitan Sewerage Commission of New York estimated the

TABLE 21.—WEIGHT OF SOLID AND LIQUID EXCREMENTS, AVERAGE URBAN POPULATION

(Based on investigations of Wolff and Lehmann)

Number, sex and age of persons contributing	Feces		Urine	
	Pounds per 100,000 persons per year	Grams per person per day	Pounds per 100,000 persons per year	Grams per person per day
37,610 men	4,521,664	150	45,217,782	1,500
34,630 women	1,237,040	45	37,458,512	1,345
14,060 boys	1,239,504	110	6,423,670	570
13,700 girls	274,736	25	5,041,344	460
Totals and means.	7,272,944	90	94,141,308	1,170

TABLE 22.—AVERAGE COMPOSITION OF HUMAN EXCREMENTS

(Based on investigations of Wolff and Lehmann)

Kind	Water	Total solids	Organic matter	Inorganic matter	Nitrogen	Phosphoric acid	Potash	Lime	Magnesia
	Per cent.								
Fresh human feces..	77.2	22.8	19.8	3.0	1.00	1.10	0.25	0.62	0.36
Fresh human urine..	96.3	3.7	2.4	1.3	0.60	0.17	0.20	0.02	0.02
Mixture of the two[1].	95.0	5.0	3.6	1.4	0.63	0.24	0.20	0.06	0.04
	Grams per capita per day								
Fresh human feces..	69.5	20.5	17.8	2.7	0.90	0.99	0.22	0.56	0.32
Fresh human urine..	1127.0	43.3	28.1	14.9	7.04	1.99	2.34	0.23	0.23
Mixture of the two.	1196.5	63.8	45.9	17.6	7.94	2.98	2.56	0.79	0.55

[1] Recalculated from a table by Rafter and Baker, to make these quantities comparable with those in Table 21, also from Rafter and Baker.

quantity of paper in sewage at 20 grams per capita per day (1910 report, page 431).

Kitchen sinks contribute soap, grease, extracts of meats and vegetables, sugar, salt, milk, and the waste food washed from dishes. Sometimes substantial quantities of garbage are improperly thrown into water-closets. The laundry and baths contribute soapy water containing starch, dissolved and suspended impurities from clothing, and excretions and tissue from the body.

Many of the better stables have drains which discharge into the sewers most of the urine and much fecal matter from animals. Carriage washstands contribute large quantities of water more or less heavily laden with mineral matter, some organic matter, oil and grease.

Waste water used for floor washing in dwellings and office buildings is usually very foul, and the washings from cuspidors may contribute pathogenic organisms. Hospital sewage often contains pathogenic organisms from the excreta as well as from other discharges from patients.

From the foregoing considerations, and using the Worcester figures, it appears that the contributions from water supply, ground water, urine, feces and paper would be as follows:

Water, total solids.......	12.7	grams per capita per day
Feces, total solids........	20.5	grams per capita per day
Urine, total solids........	43.3	grams per capita per day
Toilet and news paper....	20.0	grams per capita per day
Total...............	96.5	grams per capita per day

If this quantity be deducted from the average quantity of total solids in the sewage of 5 small residential towns in Massachusetts uninfluenced by industrial wastes or storm water, the quantity of solids derived from the sinks, baths, laundries, stables, etc., may be approximated as follows:

Total solids in sewage..................	174.0[1] grams per capita per day
Contribution from water supply, feces, urine and paper......................	96.5 grams per capita per day
Contribution from sinks, laundries, baths, and other wash waters................	77.5 grams per capita per day

Industrial wastes vary very greatly in quantity and quality. An investigation made in 1912–13 by the authors at Cincinnati for Henry M. Waite, City Engineer, and Howard S. Morse, Engineer in Charge of Sewers, indicated that these wastes amounted in 1912–13 to about 700 grams total solids, 200 grams suspended solids, 22 grams organic nitrogen and 180 grams oxygen consumed per capita per day.

One tannery in a Massachusetts town of about 9000 population produced a daily average of 350,000 gal. of wastes containing 4978 p.p.m. total solids, equivalent to about 730 grams per capita per day, a quantity far in excess of that in the normal domestic sewage of the town.

An investigation at Fitchburg, Mass., made by the authors for David A. Hartwell, Chief Eng. of the Sewage Disposal Commission, showed that the total mill solids which would be discharged into the city sewers if the wastes were admitted would be about 3500 grams per capita per day.

These cases are probably exceptional, and for the sake of illustration sewage from a city having an ordinary proportion of industries discharging liquid wastes will be assumed to contain at least 200 grams of total solids per capita daily from this source.

Storm water washes much mineral and some organic matter into the sewers. Paved streets subject to heavy horse-drawn traffic may contribute more organic matter than inorganic. The concensus of British opinion (Royal Commission on Sewage Disposal, Fifth Report, Appendix II, page 12) seems to be that the suspended solids in a unit volume are greatly increased during the first portion of a storm and may be considerably decreased after a long-continued rain has washed the streets and flushed the sewers. Instances of this are given in Chapter VIII. Watson and O'Shaughnessy observed the suspended solids in the Birmingham sewage rise from 380 to 3800 p.p.m. within an hour, owing to a sudden storm.

The Metropolitan Sewerage Commission of New York (1910 report,

[1] From Table 48, page 182.

page 431) estimated the suspended solids in street wash at 20 grams per capita per day. The average quantity may be estimated with reasonable accuracy for illustrative purposes at 25 grams per capita per day, somewhat less than Johnson's determination of these solids, 28.5 grams, at Columbus, Ohio, in 1905.

To recapitulate, a rough estimate of the total solids in the several constituents of sewage is presented in Table 23.

TABLE 23.—ROUGH ESTIMATE OF TOTAL SOLIDS IN THE SEVERAL CONSTITUENTS OF SEWAGE

Constituents	Grams per capita per day	
	Items	Totals
Water supplies and ground water (assuming soft water).	12.7	....
Feces....................................	20.5	....
Urine....................................	43.3	....
Toilet and news paper (suspended)...........	20.0	....
Sinks, baths, laundries and other domestic wash waters.	77.5	....
Total for residential sewage from separate system.	....	174.0
Industrial wastes.........................	200.0	....
Total from industrial city with separate sewers.	....	374.0
Storm water..............................	25.0	....
Total from industrial city with combined sewers.	....	399.0

Organic Matter in Sewage.—The organic matter may be said, in a general way, to consist of urea and proteids, which include most nitrogenous organic matter, and of carbohydrates, including cellulose, woody fiber, fats and soap. Of these, the most important are the proteids and urea, because they are most likely to cause offensive conditions. In filtration problems the fats become of importance if present in large quantities, as they are very stable. The general relations of these ingredients are shown in Table 24, giving a hypothetical analysis typical of American sewage.

Of these constituents most attention has been given to the nitrogenous

compounds, usually considered the active agencies in producing offensive conditions. Dr. W. P. Dunbar has stated that too much importance had been placed upon them, because nitrogen does not enter into the composition of offensive gases like sulphur, and more attention should be given to the latter. In Table 24 Winslow and Phelps estimated that about one-third of the total organic matter is nitrogenous and of this only 10 per cent. is nitrogen, whereas 50 per cent. is carbon.

TABLE 24.—COMPOSITION OF TYPICAL AMERICAN SEWAGE, IN PARTS PER MILLION

("Purification of Boston Sewage," Winslow and Phelps, page 15)

	Total	In solution	In suspension
Residue on evaporation	800	500	300
Mineral matter or ash	400	300	100
Organic or volatile matter	400	200	200
Nitrogenous matter	150	......	......
Nitrogen	15	......	......
Carbon	75	......	......
Hydrogen, oxygen, sulphur, phosphorus, etc.	60	......	......
Non-nitrogenous matter	250	......	......
Fats, etc	50	......	......
Carbon	35	......	......
Hydrogen, oxygen	15	......	......
Carbohydrates	200	......	......
Carbon	90	......	......
Hydrogen, oxygen, etc	110	......	......
Total carbon	200	......	......
Total nitrogen	15	......	......
Total hydrogen, oxygen, sulphur, phosphorus, etc.	185	......	......

Mineral Matter in Sewage.—The mineral substances consist principally of calcium and magnesium carbonates and sulphates, mineral constituents of the organic matter, many inorganic industrial wastes, and, especially where combined sewers are used, the sand, clay and other soil washed into the sewers in times of storm. The mineral content of domestic sewage is rarely important.

Certain inorganic substances may greatly increase the danger of odors, page 73. If sea water, which contains sulphates, finds its way into a sewer, particularly with bad ventilation and stale sewage, concrete surfaces may be injured by the formation of hydrogen sulphide. Thus for New Bedford, Mass., the authors recommended adding 2 in. to the thickness of the walls of a concrete intercepting sewer where subject to

tidal flooding, as a precaution in case disintegration should take place on the inner surface.

The mineral matter from industrial wastes may have a much more serious effect upon the treatment of the sewage than the mineral matter from the water supply and ground water by interfering with bacterial activities. The diluting and neutralizing effect of municipal sewage upon acid and other industrial wastes, however, often makes it practicable to treat sewage containing large quantities of them by the usual bacterial methods. Sulphate of iron may cause very large and troublesome sludge deposits where chemical treatment is necessary, or it may furnish the necessary coagulant, thus proving advantageous. It may have an unfavorable effect upon filtration through sand beds. Tannery wastes may react with other ingredients of the sewage, causing troublesome sludge deposits. Industrial wastes also frequently contain the objectionable sulphides already mentioned. Occasionally industrial wastes have contained such large quantities of arsenic or other germicides that the disinfecting action upon the organisms in filters has been unfavorable.

SEWAGE ANALYSES

The data given in Tables 45 to 48, at the end of this chapter, are mostly from actual analyses, and are grouped according to the characteristics of the cities producing the sewage. Their chief value is illustrative and as guides in making estimates of the probable quality of sewage from a community of known size and character. All data upon which the tables are based are not strictly comparable, and there was much diversity in the methods of taking the samples analyzed. Where they were taken once or twice an hour throughout the entire 24 hours of the day, 7 days a week, for long periods of time, the averages represent closely the general character of the sewage; but where isolated samples are taken at relatively long intervals, it is uncertain whether they fairly represent the sewages. Differences in methods of analysis also have an effect upon the results, and there is an ever present difficulty in obtaining proper proportions of coarse suspended matter. It is of the utmost importance, when interpreting analyses of sewage, to procure as complete information as possible regarding conditions affecting the composition of the sewage. This will often lead to discarding analyses which otherwise would have been accepted, and sometimes such information may make the results of certain analyses of unexpected value. Engineers and chemists should, therefore, be very careful when reporting analyses, to give in great detail all available data relating to the production, collection, sampling and analysis of the sewage and effluents.

Grams per Capita.—The different amounts of diluting water in sewages result in the average analysis of the sewage of one community

being sometimes of little help in predicting the composition of another sewage. If, in addition to accurate analyses, however, the quantity of sewage and contributory population are known, it is possible to calculate the weight per capita of each chemical constituent, and from such results reasonably accurate estimates of the composition of sewage of other similar places can be made. It should be noted, however, that ordinarily even by this method, the prediction of the composition is very uncertain because of lack of accurate data. There is greater accuracy in predicting the character of sewage from a residential city than from an industrial community, for in the latter the effect of trade wastes is extremely difficult to estimate without studies in detail of the quantity and quality of such wastes. Another element of uncertainty is the influence of solids in water supplies and in ground water, for they may have an important influence upon the analyses of the sewage.

Parts per million can be converted to grams per capita or the converse, by means of the following formulas, in which A = parts per million of any constitutent and C = U. S. or imperial gallons or liters per capita per day:

For United States gallons:
Grams per capita per day $= 3.785AC/1000$
For Imperial gallons:
Grams per capita per day $= 4.543AC/1000$
For liters:
Grams per capita per day $= 0.001AC$

Tables 45 to 49 show that even within groups of cities of presumably similar characteristics, there is an apparent variation in the quantity per capita of different constituents. This is because it is difficult to obtain reliable estimates of the population tributary to the sewers; errors in gaging the flow may be considerable; and errors in sampling are difficult to avoid. There are also differences due to varying characteristics of the cities and their inhabitants, which are real and not apparent.

Comparison of Sewage from Various Types of Cities.—The averages of the composition of sewage from the several classes of cities are brought together in Table 52, so that they may be readily compared. They indicate that the sewage from the large cities is, on the whole, weaker,[1]

[1] There is no generally recognized standard by which a sewage may be measured and designated as strong, or weak, although it is common to speak of sewage in these terms. In most sewage disposal problems, the organic content is more important than the mineral matter in sewage, which is usually designated as strong or weak according as it contains much or little organic matter. It is usual to have in mind the results of the several tests for albuminoid ammonia, total organic nitrogen, oxygen consumed and volatile solids. While in many cases these determinations may run substantially parallel, it is not unusual to obtain higher total organic nitrogen, for example, in one sample than in another which is higher in oxygen consumed.

The Massachusetts State Board of Health has sometimes tabulated results of analyses in the order of increasing or decreasing albuminoid ammonia. Where this determination is

or more dilute, than that from either the manufacturing or residentia communities. The weakness of the sewage of large cities, shown in Table 45, is due to the very large quantity per capita. Table 25 contains average analyses of typical strong, medium and weak sewages, which would be graded in the same order by any one of the determinations. A city may produce a strong sewage during the middle of a very dry day and a weak sewage at night or during the spring when ground water is high.

TABLE 25.—TYPICAL MASSACHUSETTS STRONG, MEDIUM AND WEAK SEWAGES

(Parts per million)

Character of sewage	Strong	Medium	Weak
Place	Brockton	Worcester	Stockbridge
Nitrogen as			
Free ammonia	61.2	20.3	12.4
Albuminoid ammonia	22.1	7.1	3.7
Total organic nitrogen	50.2	18.0	7.1
Oxygen consumed	247.0	193.0	26.8
Total solids	1584.0	882.0	298.0
Volatile solids	1074.0	429.0	177.0
Suspended solids	892.0	276.0	99.0
Chlorine	166.0	114.9	18.0

The sewage from manufacturing cities appears to be the strongest. This is due to the moderate quantity of sewage produced and the influence of industrial wastes. While in all large cities great quantities of such wastes are produced, they may not have as great an influence as in smaller manufacturing cities where a few large industries may produce a much greater quantity of wastes in proportion to the population.

The sewage of a community tends to increase in strength with the increase in the population. The small town often builds its first sewers under conditions favorable to much infiltration, which has considerable diluting effect because of the sparse population tributary to the sewers. In the larger community the number of persons tributary to a unit length of sewer is greater, but there is no corresponding increase in infiltration. In still larger communities, industrial wastes are more

made, it may well be taken as a standard of strength, unless the quality of the sewage is much influenced by industrial wastes. The nitrogen as organic nitrogen may also be used as a measure of strength, and will no doubt be considered preferable to albuminoid ammonia by some. There is now a tendency toward adopting as a measure of strength the quantity of oxygen required for their oxidation, but data are not yet available for tabulation of sewages upon this basis.

common, tending toward the still greater increase in strength. This tendency would also extend to the very large cities were it not for the fact that in many cases they use larger quantities of water per capita than those used by the small and moderate-size communities. If present tendencies toward reducing the consumption of water continue and the consumption in the large cities is reduced materially, it is probable that the strength of the sewage from such cities will be much greater than that of cities of other types, because of the density of population and the prevalence of industrial wastes.

Examination of the results of analyses expressed in grams per capita per day shows that the sewage from the large cities carries a far greater aggregate quantity of impurities, as measured by solids, than that from any other type of community. For example, in the sewage from the large cities the total solids amount to 567 grams per capita, whereas in the manufacturing and rural communities they are but 266 and 174 grams, respectively, but one-half and one-third of the first amount.

American and English Sewages Compared.—The analyses of English sewages, Table 49, show them to be materially stronger than average American sewage. This is due primarily to the more restricted use of water, and to the concentration of population and industries in large cities in England. The smaller quantity of ground water in the English than in the American sewers, due to differences in rainfall and permeability of the soil, may have a material effect. Workmanship and inspection are better in some places in England than in the United States.

The quantities of the various ingredients of sewage, expressed in grams per capita, are less in English than in American sewage. The data are so general in nature and so little information is available regarding specific cases, that it is not possible to give an accurate explanation of this fact, but it may result in part from the greater wastefulness of the American population, which tends to the discharge of much kitchen waste into the sewers, and also to the greater care of the English manufacturers in the recovery of by-products which are wasted by many American manufacturers.

Analyses of German Sewages.—The analyses given in Tables 50 and 51 were procured for the authors by Dr. D. W. Bach, Chemist of the Emschergenossenschaft. Those in Table 50 are of the sewage from communities within the Emscher District, the analyses being made under the immediate direction of Dr. Bach, who limited the selection to certain cities where he was sure the data had been secured in a trustworthy manner.

The German cities, like those in England, appear to produce very much smaller quantities of sewage per capita than American cities. There are, however, some exceptions, notably Essen, where the quantity

is given as 138 gal. per capita of total population, which is as much as is produced by many American cities of moderate size.

Analyses of French Sewage.—The analyses given in Table 53 were secured for the authors by M. Le Couppey de la Forest. They should be used with caution for, notwithstanding personal correspondence, the difficulty of obtaining representative analyses and of acquiring knowledge of local conditions was found to be very great. This fact was emphasized by M. Rolants, Laboratory Chief at the Pasteur Institute of Lille, who furnished data to M. Le Couppey de la Forest. It will be noticed in most cases that the sewage contains very little nightsoil (*matières de vidanges*). The authors were not able to obtain data relating to the quantity of sewage produced, hence it is not possible to transform these analyses into grams per capita. From such fragmentary information, it is difficult to draw any comparison between French and other sewages. That taken at La Madelaine-lez-Lille (Nord) in May, 1905, appears to have been high in solids but low in nitrogenous matter, which is to be expected of a sewage containing little nightsoil. Analyses from the three remaining places contain free ammonia in amounts fairly comparable with that of the sewage from other countries.

SUSPENDED MATTER

In chemical analyses all solids floating or suspended in sewage are reported as suspended solids. As stated in detail in Chapter II the quantity of this material may be determined by weight or by volume; in the latter case only such solids as will settle in a reasonably short time, generally 2 to 4 hours and occasionally 24 hours, are reported and

TABLE 26.—RELATIVE QUANTITIES OF ORGANIC AND MINERAL MATTER IN SOLUTION AND IN SUSPENSION

Character of solids	Worcester, Mass.	Columbus, Ohio	Large cities combined sewers	Industrial cities of medium size	Small industrial Massachusetts towns	Residential and rural Massachusetts communities
Total solids, p.p.m.	882	996	1355	1058	730	603
Dissolved solids, per cent.	69	79	78	58	67	43
Suspended solids, per cent.	31	21	22	42	33	57
Organic solids, per cent.	49	19	33	60	61	65
Mineral solids, per cent.	51	81	67	40	39	35
Organic solids, p.p.m.						
Dissolved solids, per cent.	59	57	53	43	55	34
Suspended solids, per cent.	41	43	47	57	45	66
Mineral solids, p.p.m.						
Dissolved solids, per cent.	78	84	90	80	86	61
Suspended solids, per cent.	22	16	10	20	14	39

Note.—Computed from Tables 45 to 49, inclusive, of average of analyses.

are referred to practically as the settling solids. The proportion of matter floating or in suspension is generally less than one-half of the total solids and may be taken as one-third, on the average.

Suspended matter varies greatly in size and specific gravity. Much of it is coarse, like fruit skins, matches, corks and paper. The finer portions are similar in nature to the coarser, and come in part from their breaking up during passage through sewers, screens and pumps, and in part from disintegration of the larger masses by organisms and their enzymes. As a rule the older the sewage, the more finely divided the suspended matter.

Some matters are thrown out of solution by chemical changes due to combinations of soaps, carbonic acid, ammonia and, where industrial wastes are discharged into the sewers, lime, iron salts and many other spent chemicals. When considering analyses in which the quantities of suspended solids are given it should be borne in mind that it is impossible to take samples which represent fairly the coarser particles. Where suspended matter is removed from the sewage by grit chambers or sedimentation tanks, the quantity and character of the material so removed may be more accurately determined by careful measurements and analyses of the sludge produced than by analyses, however carefully made, of the sewage and effluent.

DISSOLVED MATTER

The relative quantity of dissolved matter in sewages is on the average perhaps about two-thirds of the total solids. It includes all substances readily soluble in water, such as those contained in urine, meat and vegetable extracts, and a large variety of soluble industrial wastes, such as spent dyes, acid liquors and tanning solutions. In seaboard cities salt water, which contains about 36,000 p.p.m. of total solids, often gains admission to the sewers and has a material effect upon the quality of the sewage, increasing its mineral content and diluting the organic matter. Of the organic matter one-third to one-half is in solution.

The proportion of dissolved to suspended matter is not always constant, even in the same sewage, as chemical, physical, and bacterial action is constantly going on, converting dissolved into suspended matter and the reverse. An approximate idea of the average proportions of the organic and mineral matter in solution and suspension in sewage may be obtained from Table 26. Great caution should be exercised in drawing conclusions from the results of analyses upon which this table is based. However, there is apparently a much larger proportion of dissolved solids in the sewage of the larger cities where, as at Columbus, the hardness of the water supply and ground water is high, or where industrial wastes play an important part, as at Worcester, or where the sewage is

collected by extensive sewerage systems, resulting in conditions tending to throw much of the suspended organic matter into solution. On the other hand, in small towns a much smaller proportion of the solids is in solution, and in fact suspended matter predominates, whereas in large cities dissolved matter appears to predominate. In the small towns to which the data apply, the water supplies and ground water are relatively soft. Again, in comparing the proportion of organic and mineral solids, it appears that in Columbus, where the water is hard, the mineral matter constitutes a very large proportion of the total solids, while in the small Massachusetts towns, where the water is soft and there are no industrial wastes, the organic solids are nearly twice as great as the mineral solids. It will be seen that in all cases the proportion of dissolved mineral matter is far in excess of that in suspension.

SETTLING SOLIDS

The suspended matter in sewage may cause deposits in bodies of water into which sewage is discharged, and it may clog filters. It is necessary in many cases, therefore, to reduce the settling solids by screening, which is capable of taking out coarse matter, or by sedimentation, by which a greater proportion may be removed.

Much more study has been given to the determination of settling solids in recent years than formerly, largely because of the attention directed to its importance by Dr. Karl Imhoff. Formerly the total suspended solids were usually determined by weighing the solids retained on filter paper, or by the difference in the total residue of the sewage before and after filtering, as described in Chapter II. This does not give the quantity of solids capable of settling under working conditions. It is important to determine not only the total quantity of suspended solids, but also the quantity capable of settling.

At the sedimentation plants in the Emscher District, Imhoff found that the amount of solids settling after 2 hours' sedimentation was quite small, and he accordingly adopted 2 hours as the duration of the period in which samples should be allowed to settle under laboratory conditions in order to determine the amount of settling solids in them. He has used the volumetric method (Chapter II, page 51) for determining these solids. The data given in Table 27, furnished by Bach and Spillner, are from plants in the Emscher District.

In 1905, Geo. A. Johnson ascertained the quantity of suspended solids in the Columbus sewage which were capable of settling in different periods of time (Table 28). During these experiments, the sewage flowed continuously through the tanks, so that the results are not strictly comparable with results obtained in similar periods of time with quiescent sewage. The sewage first passed through grit chambers, but the period

of sedimentation in them was so much shorter than in the tanks that no serious error was probably introduced by attributing the precipitation of suspended matter to sedimentation in the period required for flow through the sedimentation tanks.

TABLE 27.—TOTAL SUSPENDED AND SETTLING SOLIDS IN SEWAGES IN EMSCHER DISTRICT

Place	Total suspended solids, p.p.m.	Settling solids, cc. per liter	Ratio of two determinations
Aplerbeck	546	3.88	140.8
Bochum	458	3.64	126.0
Essen Frohnhausen	925	7.60	121.8
Essen Nord	751	14.80	50.7
Essen Nordwest	898	7.10	126.4
Holzwickede	400	4.00	100.0
Lütgen-Dortmund	1024	6.69	153.1
Recklinghausen	440	5.28	83.3
Zechenkolonie Graf Schwerin	504	5.10	98.8
Sodingen	225	10.70	21.0
Teutoburgia	491	5.00	98.2
Suderwich	694	4.90	141.6
Westhausen	456	3.15	144.8

Note.—Information concerning the removal of suspended solids by grit chambers, screens and tanks at many of these places is given in Table 95, page 440.

The quantity of settling solids in sewage depends upon its strength, the character of its suspended matter, the detention period, and other more or less important conditions. The determination of settling solids, described in Chapter II, is made by allowing a sample to remain quies-

TABLE 28.—SETTLING SOLIDS IN COLUMBUS SEWAGE

Tank	Detention period, hours	Total suspended solids, p.p.m.		Solids settling in specified time, p.p.m.	Suspended solids removed (per cent.)	Volatile suspended solids, p.p.m.		Volatile solids settling in specified time, p.p.m.	Volatile suspended solids removed, (per cent.)
		Influent	Effluent			Influent	Effluent		
Grit chamber B	0.3	242	188	54	22	74	62	12	16
Grit chamber A	1.3	196	129	67	34	87	60	27	31
Sedimentation tank B	6.0	211[1]	80	131	62	85[1]	30	55	65
Sedimentation tank A	8.0	210[1]	73	137	65	78[1]	35	43	55

[1] In influent to grit chambers through which sewage passed before entering sedimentation tanks.

cent for a definite period of time, and measuring or weighing the precipitate. When sewage is settled on a large scale, however, it is generally allowed to flow continuously through tanks, which sometimes are not covered. The velocity due to flow, the wind, and temperature circulation interfere with sedimentation. It is to be expected, therefore, that the laboratory tests will give somewhat higher results than can be attained in practice, and Imhoff pointed out in 1909 that dilute sewage shows a lower percentage of separation than concentrated sewage.

TABLE 29.—SUSPENDED SOLIDS IN CHICAGO SEWAGE AND STOCK YARDS SEWAGE CAPABLE OF SETTLING IN PERIODS STATED

Kind of sewage	No. of tests	Total susp. solids	1 hour		2 hours		4 hours		12 hours	
			p.p.m	Per cent.	p.p.m	Per cent.	p.p.m	Per cent.	p.p.m	Per cent.
	Total settling solids									
Stock yards......	3	1620–1930								
	Av.	1775	1508	85	1580	89	1597	90	1633	92
Stock yards......	6	760–1070								
	Av.	915	650	71	686	75	705	77	714	78
Stock yards......	1	560	297	53	314	56	314	56	319	57
39th St..........	3	206–266								
	Av.	236	130	55	146	62	165	70	186	79
39th St..........	5	118–194								
	Av.	156	70	45	84	54	98	63	109	70
39th St..........	2	68–76								
	Av.	72	17	23	22	31	31	43	42	59
	Volatile settling solids									
Stock yards......	1	1140	889	78	935	82	980	86	1003	88
Stock yards......	4	620–840								
	Av.	730	526	72	569	78	591	81	620	85
Stock yards......	5	420–560								
	Av.	490	299	61	304	62	314	64	353	72
Cologne, Germany	7	233–447								
	Av.	340	221	65	231	68	245	72	265	78
39th St..........	8	84–127								
	Av.	106	43	41	52	49	61	58	67	63
39th St..........	2	50–52								
	Av.	51	9	18	12	24	14	27	18	35

Note.—From "Report on Industrial Wastes from the Stock Yards and Packingtown in Chicago," 1914, by Geo. M. Wisner, Chief Eng., and Langdon Pearse, Division Eng. (page 172). Tests made in can 2 ft. diam. and 9 ft. deep. Samples withdrawn from center of can 18 in. below surface of sewage. Can was filled from bottom to depth of 8 ft. 6 in. and allowed to stand quiescent for period stated (page 169).

The effect of the strength of the sewage and the length of the period of quiescent settling upon the quantity of solid matter reported as

settling solids is shown by experiments by George M. Wisner, Chief Engineer, and Langdon Pearse, Division Engineer, Sanitary District of Chicago, upon sewage and industrial wastes at Chicago. This information is summarized in Chapter XI. In a general way, the ratio of settling solids to total suspended solids increases with the increase in the strength of the sewage. It is important, however, not to be misled by such ratios, for the tests indicate that it was necessary to settle the strongest 39th Street sewage 12 hours to reduce the suspended matters in the effluent to the same quantity as that in the effluent from the weaker sewage after 2 hours. It is often important to consider the amount of solids remaining as well as the percentage removed. The quantity of solids capable of settling in different periods of time in the Chicago Stock Yards wastes and sewage is given in Table 29.

The settling solids in the Providence sewage are determined by Julius W. Bugbee, superintendent and chemist of disposal works, as stated on page 51, results obtained in this way in 1913 being given in Table 30. It is significant that the solids capable of settling in 4 hours amount to but 15 per cent. of the total solids of the sewage, although equivalent to 55 per cent. of the suspended matter.

TABLE 30.—TOTAL SUSPENDED SOLIDS AND ALBUMINOID AMMONIA IN PROVIDENCE SEWAGE, 1913

(Julius W. Bugbee in Report of City Engineer, 1913, page 46)

Designation	Parts per million	Percentage of total solids	Percentage of suspended solids
Total solids	1155.0	100	....
Suspended solids	314.0	27	100
Settling solids (4 hours)	171.0	15	55
Total albuminoid ammonia	8.46	100	....
Suspended albuminoid ammonia	5.18	61	100
Settling albuminoid ammonia	2.27	27	44

At Columbus it was found that only about 60 per cent. of the suspended matter could be removed by any economical period of sedimentation, Table 31. As the period of sedimentation and length of travel of the sewage are increased, they rapidly approach points where additional time has but little effect in removing the residual suspended matter. Assuming Columbus sewage to have averaged 210 p.p.m. suspended solids (Table 45), 84 p.p.m. (40 per cent.) were so finely divided that they failed to settle in a period of 4.2 hours while traversing a distance of 200 ft. and might be classed as colloids, to which should be added the true colloids to arrive at the total.

Table 32 summarizes information furnished to the authors by Charles

TABLE 31.—PROPORTION OF SUSPENDED SOLIDS CAPABLE OF SETTLING IN STATED PERIODS OF CONTINUOUS FLOW

(Sewage Purification at Columbus, Johnson, 1905, page 108)

Length of travel, feet	Period of flow through tank, hours	Suspended solids removed, per cent.	Suspended solids remaining, per cent.
40	0.8	35	65
80	1.7	50	50
120	2.5	55	45
160	3.3	57	43
200	4.2	60	40

C. Hommon regarding the settling and suspended solids in the sewage of Atlanta. While the Gooch crucible determinations indicate a possible removal of 81 and 51 per cent. respectively, of the total suspended solids, by sedimentation in the Imhoff tanks, Hommon stated that the percentage removal of settleable solids, as determined by the Imhoff settling glasses, is practically 100. He also stated that the 60 and 56 p.p.m. total suspended solids in the Imhoff tank effluents are nearly all colloidal material, incapable of settling.

TABLE 32.—SETTLING AND SUSPENDED SOLIDS IN ATLANTA SEWAGE

(Parts per million)

Plant	No. of samples	Total suspended solids[1]	Settling solids	Suspended solids[2]
Proctor Creek plant..........	250	320	260	60
Peachtree Creek plant.......	200	114	58	56

[1] By Gooch crucible method. [2] Total suspended solids by Gooch method in tank effluent.

The volume of solids settling out of average sewage at Worcester, Mass., in 2 hours, as determined by the conical glass method, is given in Table 33. There was a noticeable error in the observed volume, due to voids in the settled material, so a correction, often quite large, was made by subtracting the estimated volume of voids from the actual reading. There was a marked variation in the volume of settling solids during each period, but the averages followed in a general way the concentration of the sewage.

At the suggestion of the authors, Roy S. Lanphear, supervising chemist of the Worcester Sewer Department, investigated the relation of the volume of settling solids determined by the conical glass method to the true settling solids by weight. The weight of solids settling in 2 hours in many cases constituted a very large percentage of the total suspended matter in the sewage, as determined by the filter-paper

method, which is explained in part by the precipitation of colloidal matters. The tests were made with strong day sewage. The settling solids bore no definite relation to the total suspended matter, and there was a much wider variation in the settling solids by volume than by weight. In the case of several samples, compacting of the sludge after 2 hours more than offset the increase in volume due to additional settling solids.

TABLE 33.—SETTLING SOLIDS[1] IN AVERAGE SEWAGE AT WORCESTER, MASS., BY THE CONICAL GLASS METHOD

Period 1913–1914	Number of samples	Corrected volume of settling solids in cc. per liter			Percentage actual reading was in excess of true volume		
		Maximum	Minimum	Average	Maximum	Minimum	Average
Apr. 21–30..	37	13.50	1.55	5.44	212	1.0	32.4
June 13–25..	37	29.01	3.40	10.03	132	0.5	23.2
Nov. 4–10...	24	19.20	2.65	9.07	60	1.0	14.1
Feb. 17–26..	45	22.52	2.90	8.72	193	2.1	44.6
Average.....	....	21.06	2.62	8.31	149	1.1	28.6

The erratic character of the conical glass method was shown by the results obtained from simultaneous measurements with 4 glasses for each sample. The determinations showed in many cases a wide variation in the actual readings of settling solids, but the volumes corrected for voids were much more uniform. The ratio between weight and volume of settling solids from day to day was very variable, being several times as much in some samples as in others. The volume of settling solids determined in this way was not a measure of the volume of sludge that would be deposited in tanks, owing to the low, variable density of the settling solids in the conical glass.

COLLOIDAL MATTER IN SEWAGE

In Chapter II colloids were stated to be substances in a sort of pseudo-solution which would not diffuse through a parchment diaphragm; for practical purposes the colloids may be held to include those fine suspended particles usually reported as suspended solids but incapable of settling in sedimentation tanks as ordinarily operated.

F. R. O'Shaughnessy (Inst. San. Engrs., Feb. 16, 1914) stated that 80 per cent. of the dissolved matter in water vigorously shaken with fecal matter was colloidal. The following analysis shows how great a proportion of the organic and oxygen-consuming matter was in this colloidal condition:

[1] For 2-hour period of sedimentation.

Dissolved solids................	Total	2710.0 parts per 1,000,000
Colloidal matter...............	Total	2260.0 parts per 1,000,000
	Volatile	2125.0 parts per 1,000,000
Oxygen absorbed in 4 hours at 80°F.	Total	547.3 parts per 1,000,000
	Colloids	411.6 parts per 1,000,000
	Non-colloids	135.7 parts per 1,000,000

It is generally agreed that agitation, such as that caused by flowing very rapidly through a long sewer, over steps or through screens and by pumping, changes some suspended matter in sewage into colloids. This is illustrated by an experiment at Leeds, England, cited by O'Shaughnessy. The average of three somewhat erratic experiments showed that 30 per cent. of the albuminoid ammonia was in form of colloids before, and 42.5 per cent. after pumping; and that 14.9 per cent. of the organic matter as indicated by the oxygen consumed was colloidal before and 17.2 per cent. after pumping. The colloids in the screened sewages before and after pumping are given in Table 34.

TABLE 34.—COLLOIDAL MATTER IN LEEDS SEWAGE BEFORE AND AFTER PUMPING

(Parts per million)

	Albuminoid ammonia		Oxygen consumed, 4 hours	
	Before pumping	After pumping	Before pumping	After pumping
Test 1.............	2.40	4.27	21.8	19.3
Test 2.............	4.65	4.95	23.2	32.8
Test 3.............	1.47	0.87	20.0	22.4
Average............	2.84	3.36	21.7	24.8

At the Philadelphia Sewage Experiment Station, many tests were made in 1910 to ascertain the quantity of organic matter in the sewage as indicated by oxygen consumed, in suspension and capable of settling when allowed to stand quiescent in a bottle for 2 hours, in colloidal condition, and dissolved. The averages of the results obtained from August to April, inclusive, are as follows:[1]

[1] For determination of oxygen consumed, samples were acidified at room temperature and placed in water bath at 100°C. for 30 minutes (report, page 27). Colloidal matter was precipitated by Fowler's clarification method. This method is to add to 200 cc. of the sewage 2 cc. of a 5 per cent. solution of sodium acetate and 2 cc. of a 10 per cent. ferric ammonium alum solution. The sample is then brought to boiling and allowed to remain over a low flame for 2 minutes. It is then cooled and filtered. In this way the colloids are precipitated, leaving a clear filtrate which may be taken for practical purposes to contain only substances in true solution. Fowler states that "this method has been found to yield as instructive results as the method of dialysis, while it occupies much less time, and probably, in consequence, is less liable to error." (*Jour. Soc. Chem. Ind.*, vol. xxvii, page 205, 1908.)

Oxygen consumed:
Total........................... 79.9 parts per million
Settling........................ 14.9 parts per million
Colloidal....................... 28.8 parts per million
Dissolved....................... 36.2 parts per million

These determ nations indicate that about 18.5 per cent. of the organic matter was capable of settling in 2 hours and 36 per cent. was in colloidal condition, leaving about 45.5 per cent. in solution. Consideration should be given to the large quantity of industrial wastes in this sewage, the effect of which is indicated in part by the large quantity of fixed solids, 35 per cent. higher on workdays than on holidays. (Report, page 30.)

Settling Solids and Colloidal Matter in Industrial Wastes.—The quantity and character of suspended and colloidal matter in industrial wastes often impose a heavy burden upon a treatment plant. The data in Table 35 were compiled from analyses of wastes from a large

TABLE 35.—CHARACTER OF SETTLING AND DISSOLVED SOLIDS AND COLLOIDAL MATTER IN WASTES FROM A TANNERY AND WOOL-SCOURING PLANT

	Parts per million	Percentage of total solids	Percentage of dissolved solids	Percentage of suspended solids[1]
Total solids in original sample....	5,960	100.0	147.1	312.3
Total solids after settling 24 hours.	4,788	80.3	118.2	250.9
Settling solids by difference......	1,172	19.7	28.9	61.4
Solids after filtering through paper.	4,530	76.0	111.8	237.4
Additional solids removed by filter.	258	4.3	6.4	13.5
Colloids after diffusion through parchment.	478	8.0	11.8	25.1
Crystalloids or dissolved solids by difference.	4,052	68.0	100.0	212.3
Total suspended and colloidal matters.	1,908	32.0	47.1	100.0

[1] Including colloidal matter.

tannery and wool-scouring plant. A sample of paper mill washer wastes, containing about 2300 p.p.m. of total suspended matter, allowed to stand quiescent, precipitated in 12 hours 95 per cent. and in 6 hours 88 per cent. of the suspended matter capable of settling in 24 hours. After these wastes had passed through settling tanks with a period of flow of about 10 hours, they were found to contain 564 p.p.m. of suspended matter, in addition to 665 p.p.m. of colloidal matter. Sedi-

mentation tests of wastes from a woolen mill, not including wool-scouring wastes, gave the results stated in Table 36.

TABLE 36.—SETTLING SOLIDS IN CRUDE WOOLEN-MILL WASTES EXCLUDING WOOL-SCOURING WASTES

Period of sedimentation (hours)	Total solids in supernatant liquid, p.p.m.	Settling solids, p.p.m.	Percentage of removal in 24 hours	Percentage of total suspended matter removed[1]
0	807	0	0.0	0.0
1	742	65	54.6	32.3
2	728	79	66.4	39.3
4	716	91	76.5	45.3
8	705	102	85.7	50.7
24	688	119	100.0	59.2

[1] Total suspended matter 201 p.p.m.

BACTERIA

A consideration of the general composition of sewage would be incomplete without reference to its bacterial content, although this has been discussed at length in Chapter III. In general the bacteria vary from 1,000,000 to 10,000,000 per cubic centimeter. The number increases greatly as the sewage increases in age, especially during the first day or two. If a sample is kept in a bottle or basin without dilution, agitation or other agency for changing conditions, the bacteria will reach their maximum number in the course of a few days and then gradually decrease, although they will by no means entirely disappear even after very long periods of time.

EFFECT OF TEMPERATURE

The composition of sewage is affected by its temperature, which varies considerably from season to season and depends partly on geographical location. At Gloversville, N. Y., the average temperature of the sewage was found by Eddy and Vrooman in 1909 to be 53°F. (12°C.), while at Matunga, near Bombay, India (Fowler, *Engineering News*, vol. lviii, 1907, page 146), it ranged on the average from 78° to 90°F. (25° to 32°C.). In Table 37 are shown the monthly temperatures of sewage at various places in the northeastern part of the United States.

When the temperature is low, the activity of bacterial life is at a minimum, and the sewage may contain dissolved oxygen originally present in the diluting water, some nitrates and nitrites, and may remain in relatively inoffensive condition for some time. In warm weather, however, bacteria are very active and consequently the dissolved oxygen

TABLE 37.—TEMPERATURE OF SEWAGE IN AMERICAN CITIES

Month	Gloversville, N.Y., 1908-09		Lawrence, Mass.		Columbus, Ohio		Waterbury, Conn.		Philadelphia, Pa., 1914[1]	
	F.	C.	F.	C.	F.	C.	F.	C.	F.	C.
January.....	47	8.3	51	10.6	52	11.1	50	10.0	.....	......
February....	46	7.8	48	8.9	48	8.9	50	10.0	50	10.0
March......	45	7.2	47	8.3	50	10.0	49	9.4	50	10.0
April.......	46	7.8	54	12.2	58	14.4	52	11.1	52	11.1
May........	51	10.6	58	14.4	61	16.1	60	15.6	55	12.8
June	57	13.9	66	18.9	66	18.9	61	16.1	63	17.2
July........	59	15.0	68	20.0	....	......	65	18.3	65	18.3
August.....	63	17.2	71	21.7	69[2]	20.6	72	22.2	65	18.3
September..	62	16.7	66	18.9	70	21.1	73	22.8	63	17.2
October	57	13.9	64	17.8	65	18.3	66	18.9	61	16.1
November ..	52	11.1	59	15.0	61	16.1	52	11.1	59	15.0
December ..	49	9.4	48	8.9	53	11.7	50	10.0	52	11.1
Average ..	53	11.7	58	14.4	59	15.0	58	14.4	....	......

[1] Report Bureau of Sewers, 1914 (chart).
[2] For 16 days.

as well as the oxygen of the nitrites and nitrates is rapidly exhausted and putrefaction begins. The effect of this is shown in Table 38, giving the quantities of dissolved oxygen, nitrates and nitrites in sewage during different months.

TABLE 38.—DISSOLVED OXYGEN, NITRITES AND NITRATES IN BOSTON SEWAGE, 1906

(Sanitary Research Laboratory, Mass. Inst. Tech., vol. iv, 1908, page 410)

Period	February March	May June	August September	November December
Nitrates, p.p.m..........	0.4	0.3	0.0	0.1
Nitrites, p.p.m..........	0.0	0.0	0.1	0.0
Dissolved oxygen, p.p.m..	5.6	2.0	0.2	2.5

The seasonal variation of free and combined oxygen, however, is not the only change due indirectly to the temperature, and perhaps in many cases is not the most important. Generally, the quantity of oxygen in sewage is so small in proportion to its oxygen requirements that its presence may be neglected. The activity of bacteria at high summer temperatures causes important changes in the organic matter. This is shown by a decrease of the organic nitrogen and albuminoid ammonia, with an increase of free ammonia more or less closely corresponding.

Suspended matter, which in winter passes to the outfall in relatively coarse condition, may in summer be so disintegrated through bacterial activity that it will be in a much finer state of subdivision. Such a change may be very marked where deposits of organic matter are formed in sewers. During cold weather these deposits may accumulate in relatively large quantities, whereas during the higher temperatures of summer decomposition may take place so rapidly, and such large quantities of gas may be formed, that great masses of the previously accumulated sludge will rise to the surface and float along with the sewage until the gas is liberated, after which the material will be carried in suspension because of its fine subdivision.

VARIATIONS IN QUALITY OF SEWAGE

Hourly Variation.—The composition of the sewage of a community changes rapidly from time to time on account of the fluctuating quantities of constituents contributed and the entrance of more or less ground water, surface water and storm water into the sewers. The night sewage is much weaker than that of the day, and hourly analyses of sewage show that its composition depends very directly upon the activities of the population. In Table 39 are given the proportional hourly variations in flow and quantity of various ingredients of the sewage of Columbus, Ohio, which are typical of average hourly changes.

TABLE 39.—COMPARISON OF THE PERCENTAGES WHICH THE FLOW OF SEWAGE AT THE COLUMBUS OUTFALL AND THE AMOUNT OF ITS DIFFERENT CONSTITUENTS AT DIFFERENT HOURS ARE OF THE AVERAGES FOR THE DAY

("Report on Sewage Purification, Columbus, Ohio, 1905," Geo. A. Johnson, page 33)

Hour	Rate of flow	Oxygen consumed			Nitrogen as		Chlorine	Suspended matter			Fats	Bacteria per cc.
		Total	Dissolved	Suspended	Free ammonia	Organic		Total	Volatile	Fixed		
12 a.m.– 2 a.m.	88	35	42	27	73	47	72	59	29	81	33	75
2 a.m.– 4 a.m.	83	69	23	123	56	31	74	43	28	53	26	47
4 a.m.– 6 a.m.	80	28	29	27	42	23	73	37	18	52	30	36
6 a.m.– 8 a.m.	89	28	29	27	49	28	69	37	29	43	22	56
8 a.m.–10 a.m.	113	176	181	169	108	167	94	213	156	257	81	92
10 a.m.–12 m.	117	116	136	92	109	117	143	145	160	134	111	167
12 m. – 2 p.m	115	123	149	92	110	115	123	110	129	96	111	139
2 p.m.– 4 p.m.	117	117	149	81	167	126	111	110	109	112	81	167
4 p.m.– 6 p.m.	115	197	149	254	147	225	111	165	229	116	307	103
6 p.m.– 8 p.m.	103	88	81	96	99	84	101	94	109	83	163	125
8 p.m.–10 p.m.	92	90	87	92	99	80	114	54	64	46	81	106
10 p.m.–12 p.m.	89	58	68	46	84	63	80	52	50	54	74	81

Daily Variation in Quality.—Causes similar to those affecting the hourly changes make the quality of the sewage different on different days. The greatest departure from the average daily quality occurs on Sunday, when industrial wastes are not discharged into the sewers and household activities are checked. Where the week's washing is done on Monday, the quantity of sewage is increased by the discharge of wash-waters, and the soap and other ingredients affect the quality of the sewage. Analyses typical of the daily variation in composition of the Columbus sewage are given in Table 40.

TABLE 40.—PERCENTAGE WHICH AVERAGE ANALYSES OF COLUMBUS SEWAGE FOR EACH DAY ARE OF AVERAGE FOR THE WEEK, TAKING INTO ACCOUNT THE VARIATION IN THE DAILY RATE OF FLOW OF SEWAGE

(Computed from "Report on Sewage Purification, Columbus, Ohio, 1905," George A. Johnson)

1904–05, day of week	Oxygen consumed total	Nitrogen as				Chlorine	Residue on evaporation								
							Total			Volatile			Fixed		
		Total organic	Free ammonia	Nitrites	Nitrates		Total	Dissolved	Suspended	Total	Dissolved	Suspended	Total	Dissolved	Suspended
Sun.....	61	75	102	112	93	84	86	92	64	71	76	64	90	95	63
Mon....	114	101	104	89	93	100	105	101	121	118	107	131	102	100	115
Tues....	110	105	103	112	93	105	103	103	102	108	107	109	101	102	99
Wed....	106	107	100	89	93	104	101	103	93	99	102	95	101	103	93
Thur....	98	102	94	100	139	100	99	100	98	98	98	99	100	100	97
Fri.....	104	102	101	100	93	102	102	100	110	101	103	99	102	100	117
Sat.....	100	104	97	100	93	102	102	101	107	101	103	100	102	100	111

Seasonal Variation in Quality.—Sewage flowing during the spring or "wet" months is generally considerably weaker than that of summer and early autumn, owing to the storm and ground waters finding their way into the sewers. The composition is also affected to some extent by temperature and bacterial action, as well as by dilution. In Table 41 are given the proportional variations in character of sewage from month to month at Worcester, Mass.

In many places the amount of chlorine in the sewage is practically proportional to the dilution, and knowing the chlorine, the quality of the sewage and effect of dilution may be calculated.

It will be seen from the preceding statements regarding the fluctuating quality of sewage, and the same fluctuations characterize wastes, that it is necessary to obtain samples planned and taken intelligently in order to be able to obtain from them a correct knowledge of the general character of the sewage or wastes. It is generally advisable to secure a

series of such samples and to be guided by the average results of analyses, giving due consideration to variations. It is possible to be misled by either average results or analyses of isolated samples.

TABLE 41.—PERCENTAGE WHICH ANALYSES OF WORCESTER SEWAGE FOR EACH MONTH ARE OF THE AVERAGES FOR THE YEAR

(Based on a weighted average analysis for the year, taking into account the relative average flow of sewage in each month)

	Ammonia		Oxygen consumed	Chlorine	Residue on evaporation			Precipitation, inches
	Free	Total albuminoid			Total suspended	Volatile suspended	Fixed suspended	
1907								
January...	93	110	106	104	92	92	93	2.76
February..	112	109	117	94	98	115	58	1.84
March....	95	95	93	80	102	91	129	1.69
April.....	88	88	90	81	84	85	81	2.72
May......	115	103	103	103	85	91	73	2.92
June......	115	104	99	117	123	121	129	3.82
July......	127	122	122	148	148	162	115	2.55
August....	134	128	143	149	131	147	92	1.08
September.	119	114	110	125	139	118	186	9.38
October....	81	85	89	88	97	94	104	4.63
November.	68	85	79	72	74	53	122	6.06
December.	86	76	74	75	61	75	27	4.53

SEWAGE AT LAWRENCE, MASS.

Because the sewage of Lawrence has been employed in nearly all experiments of the Massachusetts State Board of Health, to which frequent reference is made in this volume, the following explanation of its character is given.

Lawrence is a very densely populated industrial city, but the sewage used at the Experiment Station is taken from a trunk sewer at a point above the entrance of mill wastes, so that it is a strictly domestic sewage. At times it contains street washings, as the sewers are built on the combined system. A 2½-in. pipe, 4300 ft. long, leads from the sewer to the Station. Analyses of sewage taken when it enters and leaves this pipe show that substantial changes take place in transit. Most of the experiments have been performed with the "station sewage," Table 42, or effluents derived from it. During much of the experimental period, determinations of solids were omitted, but they were resumed in 1902, and Table 43 gives their amount in 1910. All of these analyses

TABLE 42.—AVERAGE ANALYSES OF LAWRENCE STREET AND STATION SEWAGES

(From Reports of the Massachusetts State Board of Health, 1908, 1909, 1910)

Sewage	Street	Station
Years	1894–1910	1888–1910
Temperature:		
Degrees Fahrenheit	60.0	55.0
Degrees Centigrade	15.6	10.0
Nitrogen as:		
Free ammonia, p.p.m	22.6	32.9
Total albuminoid ammonia, p.p.m	7.8	5.9
Dissolved albuminoid ammonia, p.p.m	4.7	2.7
Chlorine, p.p.m	123.0	102.0
Nitrogen as:		
Nitrites, p.p.m	1.3	....
Nitrates, p.p.m	0.15	....
Oxygen consumed, p.p.m	86.0	45.0

were of samples taken during the day time, which are estimated by the State Board of Health to be fully twice as strong as the night sewage. (Report, 1908, page 261.)

TABLE 43.—SOLIDS IN LAWRENCE STATION SEWAGE, 1910

	Total solids	Dissolved solids	Suspended solids
Total, p.p.m	665	513	152
Volatile, p.p.m	283	166	117
Fixed, p.p.m	382	347	35

INDUSTRIAL WASTES

Among the industries producing large quantities of wastes likely to prove embarrassing in the treatment of the sewage are tanning, wool-scouring, wool-washing, cloth-washing, dyeing and bleaching, wire-drawing and galvanizing, pulp and paper-making, brewing and gas-making. Analyses of some industrial wastes are given in Table 54, at the end of the chapter. The quantities should be taken only as showing some of the constituents of wastes from different industries

and not as the actual amounts for which provision should always be made.

Quantity of Industrial Wastes.—The quantity of wastes produced depends upon the character of the industry and the size of the plant or plants. In large cities the proportion of sewage made up of industrial wastes is usually relatively small. The Milwaukee, Wis., Sewage Disposal Commission, 1911, estimated that the industrial wastes comprised 35 per cent. of the total flow of sewage. In Gloversville, N. Y., a small city where the tanning of glove leather is an important industry, there are about 20 tanneries, the total wastes from which constitute 38 per cent. of the flow of sewage (Report on Sewage Disposal Experiments, 1909, Eddy and Vrooman, page 28). At Fitchburg, Mass., the principal industries producing liquid wastes are paper-making, bleaching and dyeing, wool-scouring and washing, and cloth-washing. The quantity of industrial wastes in this city is estimated by the authors to be equivalent to 75 per cent. of the quantity of sewage and wastes together. (Report to David A. Hartwell, Chief Engineer, Sewage Disposal Commission, 1911.) The estimated quantities of industrial wastes in these cities are given in Table 44; not all of them are now discharged into the sewers.

TABLE 44.—QUANTITIES OF INDUSTRIAL WASTES IN A FEW AMERICAN CITIES

City	Kind of wastes	Gal. per capita per day	Percentage of total sewage
Fitchburg, Mass.	Paper mills	262.0	67.3
	Woolen mills	22.0	5.7
	Gingham mills	5.0	1.8
	Total	289.0	75.0
Milwaukee, Wis.	Breweries, tanneries and packing houses.	....	35.0
Gloversville, N. Y.	Tanneries	47.5	38.0
Cincinnati, Ohio[1]	Breweries, tanneries, packing houses, soap factories.	47.5	31.0

[1] Report on a Plan of Sewerage, 1913, Morse and Eddy, page 225.

Tannery Wastes.—The first process in tanning is soaking the hides, which removes some organic matter and any soluble chemical used in curing the skins. The hair is then removed in lime vats or otherwise. When lime is used, large quantities of it are carried off with the waste liquor. In tanning, large quantities of chemicals are used, in some cases nearly a pound of chemical for every pound of hides. In Gloversville, N. Y., it was found that fully one-half of the total weight of hides and

chemicals found its way into the wastes. The quantities of suspended matter in tannery wastes are so large that the latter should pass through settling tanks before entering the sewers. This will prevent the deposition of large quantities of tannery wastes (solids) in the sewers and sedimentation tanks at the disposal works.

Tannery wastes contain small numbers of bacteria, and in some cases may be actually sterile. When mixed with domestic sewage they have some disinfecting action, but unless the proportion of wastes to sewage is very large, this action will not prevent successful treatment of the combined sewage by biological methods. (Eddy and Vrooman, "Report on Sewage Disposal Experiments," Gloversville, 1909, page 11.) Tannery wastes are usually cold, and if they constitute a large proportion of the sewage of a community they may, on account of reduced temperature, affect unfavorably the process of purification.

Wastes from Wool-scouring and Washing.—Raw wool contains much dirt and grease, which are removed by scouring or washing in warm water, with alkali and soap. From 1 to 5 gal. of water per pound of wool are required for washing by modern methods, and much more by methods less economical in the use of water; 100 gal. of water to 1 lb. of wool have been noted. The waste liquid is laden with fats and suspended mineral and organic matter. Shrinkage in the weight of the wool may, by this process, amount to 60 to 75 per cent. for territory (United States) wools and 50 to 60 per cent. for English wools. The amount of grease varies from 8 to 12 per cent. for American, and 12 to 16 per cent. for English wools.

It is sometimes very important to remove the grease from the wool-washings before they are discharged into the sewage, because it is likely to interfere seriously with the process of purification. The grease discharged from a wool-scouring plant into municipal sewers has been known to seal the surface of sand filters and put the beds absolutely out of service. The removal of the grease requires the use of acids or other chemicals, which remain to a large extent in the liquors after the grease has settled out or been skimmed off. If the wastes constitute a large proportion of the municipal sewage, these chemicals should be neutralized before being discharged into the sewers, for they may have enough disinfecting action upon the sewage to render biological treatment difficult.

Woolen-cloth Washings.—These consist of soapy water used in washing cloth before dyeing, spent dyes from vats and water used in rinsing the dyed cloth. These wastes are generally cold and contain few bacteria. They are not readily purified by biological methods unless mixed with sewage. If, however, the proportion of the wastes to domestic sewage is not large, they will not seriously interfere with the purification of the sewage by biological methods.

Paper Mill Wastes.—The basis of all papers is vegetable fiber or cellulose, and a part of the paper trade wastes comes from the treatment of raw materials to rid them of resins, gums, silica, fats, oils, dirt, in fact everything of which the raw material is composed except the cellulose.

In one process of manufacture the treatment consists of boiling the stock with alkali for several hours, after which the material is washed. With some stock, beating and pulping are necessary, after which the material is bleached and then made into paper. The wastes from the boilers are small in quantity, hot, strongly alkaline, and contain the gum and dirt from the raw material. The washer wastes are similar to the boiler wastes but much more dilute, the quantity of washer wastes being frequently ten times the quantity of boiler wastes. The washer wastes contain considerable fiber, which can be removed by screening and by settling in tanks. The details of the processes at any mill affect the nature and quantity of the wastes.

In another process, chips of wood are cooked in a digester with sulphite liquor, obtained by drawing the fumes of burning sulphur through lime water by a vacuum process, or passing them through broken limestone. The digested pulp is blown into tanks and washed. The sulphite liquor from the blow tanks is a heavy, dark brown liquid containing the dissolved material from the wood, sulphite of lime, sulphurous acid and some pulp. The treatment of this waste is very difficult, and the paper industry as well as sanitary authorities have devoted much attention to it.

The machine wastes, amounting to five to ten times the quantity of washer wastes, contain the fine fiber which escapes from the paper-making machines, together with small quantities of clay, if it is used with the pulp, and coloring and coating materials where colored and coated papers are being manufactured. The fine fiber, clay and usually the coloring matter are in suspension, but so finely divided that the wastes must be dosed with some chemical, as alum, to produce rapid precipitation in tanks. This will remove 90 to 95 per cent. of the suspended matter which may be returned to the machines where the lower grades of paper are produced.

All machine wastes are cold and contain few bacteria. They are not readily purified by biological methods unless mixed with sewage, which, in many cases, is not practicable on account of the very large quantities of wastes in comparison with the sewage from the communities in which the mills are situated. In many cases, paper mill wastes may be sufficiently purified by chemical treatment, sedimentation, filtration, or combinations of these processes, to be discharged into streams without biological treatment.

TABLE 46.—ANALYSES OF SEWAGE FROM MANUFACTURING CITIES

(Parts per million, p.p.m.; grams per capita daily, g.p.c.).

	Worcester, Mass.		Gloversville, N. Y.		Waterbury, Conn.	Brockton,[6] Mass.		Clinton,[6] Mass.		Average[7]	
Date of analyses	1910		1908–9		1908	1910		1910		....	
Population (1910)	145,986		20,542		73,141	56,878		13,075		....	
Population served	136,000		16,800		....	37,000		10,000		....	
Sewage, gal. per cap. connected daily	107		154		....	35		83		....	
	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.
Nitrogen as:											
Free ammonia[1]	20.3	6.4[1]	12.0	7.0	7.8	61.2	8.2	31.4	9.9	26.5	7.8
Albuminoid ammonia	7.1	2.9	......	......	......	22.1	3.0	6.6	2.1	11.9	2.7
Organic nitrogen	18.0[2]	7.5	23.0	13.4	14.8	50.2	6.6	14.7	4.6	24.1	8.0
Nitrites	......	......	0.38	0.22	0.14	......	......	......	......	0.26	......
Nitrates	......	......	0.87	0.50	1.52	......	......	......	......	1.19	......
Oxygen consumed	135.2[3]	56.0[3]	95.0[4]	55.0[4]	46.0[4]	247.0[4]	32.7[4]	83.5[4]	26.2[4]	133.0[8]	48.0[8]
Chlorine	114.9	47.0	158.0	92.0	48.0	166.0	22.0	59.7	18.7	109.0	44.9
Alkalinity	......	......	217.0	126.0	41.0	......	......	......	......	129.0	......
Solids:											
Total	882.0	365.0	......	......	......	1584.0	210.0	707.0	222.0	1058.0	266.0
Volatile	429.0	178.0	......	......	......	1074.0	142.0	401.0	126.0	635.0	149.0
Fixed	453.0	187.0	......	......	......	510.0	68.0	306.0	96.0	423.0	117.0
Suspended solids:											
Total	276.0	114.0	406.0[5]	236.0	165.0	892.0	118.0	182.0	57.0	384.0	97.0[9]
Volatile	174.0	72.0	229.0[5]	133.0	115.0	779.0	103.0	142.0	44.0	288.0	74.0[9]
Fixed	102.0	42.0	176.0[5]	103.0	50.0	113.0	15.0	40.0	13.0	96.0	23.0[9]
Dissolved solids:											
Total	606.0	251.0	......	......	......	692.0	92.0	525.0	164.0	608.0	169.0
Volatile	255.0	106.0	......	......	......	295.0	39.0	259.0	81.0	270.0	75.0
Fixed	351.0	145.0	......	......	......	397.0	53.0	266.0	83.0	338.0	94.0
Fats	......	......	48.0	28.0	26.0	......	......	......	......	37.0	......

[1] Figures reported as "free ammonia" have been changed to read "nitrogen as free ammonia." [2] Estimate by supervising chemist, A. L. Fales. [3] Boiled 2 minutes. [4] Boiled 5 minutes. [5] Determined by Gooch crucible method. [6] The population contributing sewage is estimated by multiplying the number of house connections by a factor obtained, in part, by examining the 1903 report of the Mass. State Board of Health, and computing the number of persons per connection as there given, the total quantity of sewage being obtained from the records of the Board. [7] The average suspended matter in Worcester, Brockton and Clinton sewage was 450 p.p.m. total, 365 volatile and 85 fixed. [8] Average of results reduced as far as possible to comparable basis of "5 minutes' boiling." Analyses of sewage of Massachusetts cities were obtained from the State Board of Health by courtesy of X. H. Goodnough, Chief Eng. Brockton sewage is not much affected by industrial wastes.

TABLE 45.—ANALYSES OF SEWAGE FROM LARGE CITIES HAVING COMBINED SEWERS

(Parts per million, p.p.m.; grams per capita daily, g.p.c.)

	Chicago		Philadelphia (estimate)		Boston		Columbus		Milwaukee (estimate)		Providence		Average excluding Philadelphia and Milwaukee	
Date of analyses	1909–10		1909–10		1905–7		1904–5		1910		1909		......	
Population (1910)	2,185,283		1,549,008		670,585		181,548		375,857		224,326		......	
Population served	270,000				250,000		75,000		......		199,000		......	
Sewage, gal. per cap. connected, daily	289				200		121[6]		197		102[9]		178	
			Testing station	City estimate										
	p.p.m.	g.p.c.	p.p.m.	p.p.m.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.
Nitrogen as:														
Free ammonia[1]	8.8	9.6	4.0	4.0	13.9	10.5	11.0	5.0	12.0	9.0	15.4	6.1	10.6	7.8
Albuminoid ammonia	......	......	......	......	......	......	......	......	......	......	7.0	2.8	7.0	......
Organic nitrogen	7.6	8.3	6.3	15.0	9.1	6.9	9.0	4.1	9.0	7.0	......	......	8.0	6.4
Nitrites	0.11	0.12	0.23	0.2	0.0	0.0	0.09	0.04	......	......	......	......	0.11	0.05
Nitrates	0.35	0.38	1.00	1.0	0.2	0.15	0.20	0.09	......	......	......	......	0.44	0.21
Oxygen consumed	38.0[2]	42.0[2]	76.0[2]	100.0[2]	56.0[2]	42.4[2]	51.0[5]	23.0[5]	100.0[5]	76.0[5]	89.0[7]	35.0[7]	59.0[10]	35.6
Chlorine	40.0	44.0	39.0	80.0	2300.0[4]	......	65.0	30.0	59.0	45.0	496.0[4]	......	48.0	37.0
Alkalinity	208.0	227.0	128.0	125.0	125.0	95.0	......	......	111.0	85.0	......	......	153.6	161.0
Solids:														
Total	......	515.0[3]	......	......	......	......	996.0	456.0	588.0	450.0	1715.0[8]	678.0	1355.5	567.0
Volatile	......	......	......	......	......	......	185.0	85.0	326.0	250.0	720.5[8]	285.0	452.8	185.0
Fixed	......	......	......	......	......	......	811.0	371.0	261.0	200.0	994.5[8]	393.0	902.8	382.0
Suspended solids:														
Total	141.0	155.0	189.0	160.0	135.0	102.0	209.0	96.0	131.0	100.0	397.0[8]	157.0	214.2[11]	127.0[11]
Volatile	81.0	89.0	130.0	125.0	91.0	69.0	79.0	36.0	78.0	60.0	343.5[8]	136.0	144.9	72.0
Fixed	60.0	66.0	59.0	35.0	44.0	33.0	130.0	60.0	52.0	40.0	53.5[8]	21.0	69.3	45.0
Dissolved solids:														
Total	......	......	......	......	......	......	787.0	361.0	457.0	350.0	1318.0[8]	521.0	1052.5	441.0
Volatile	......	......	......	......	......	......	106.0	49.0	248.0	190.0	377.0[8]	149.0	241.5	99.0
Fixed	......	......	......	......	......	......	681.0	312.0	209.0	160.0	941.0[8]	372.0	811.0	342.0
Fats	23.0	25.0	28.0	40.0	......	......	25.0	11.0	52.0	40.0	......	......	25.3	25.0

[1] Figures reported originally as "free ammonia" have been computed to read "nitrogen as free ammonia." [2] Sample immersed in boiling water 30 minutes; multiply by 0.5 to convert approximately to "5-minute boiling" results. [3] Figures made from determinations between Oct. 5 and Nov. 21, 1910; average flow during this period 234 g.p.c., not included in average. [4] Contains sea water; chlorine and alkalinity from W. S. & I. Paper 185, U. S. Geol. Sur.; chlorine not included in average. [5] Boiled 5 minutes. [6] Page 34, Report on Sewage Disposal. [7] Boiled 2 minutes; divide by 0.7 to convert approximately to "5 minutes' boiling" results. [8] From "Sewage Disposal," by Kinnicutt, Winslow & Pratt; stated by Bugbee to represent sewage fairly. [9] Based on city engineer's estimate of population served. [10] Average of results reduced so far as possible to comparable basis of "5 minutes' boiling." [11] The average suspended solids at Columbus and Providence were 303 p.p.m. total, 211.3 volatile and 91.7 fixed, or 126 g.p.c. total, 86 volatile and 40 fixed.

TABLE 48.—ANALYSES OF SEWAGE FROM SMALL RURAL, RESIDENTIAL COMMUNITIES IN MASSACHUSETTS

(Parts per million, p.p.m.; grams per capita daily, g.p.c.)

	Concord		Andover		Spencer		Stockbridge		Leicester		Average	
Date of analyses	1910		1910		1910		1910		1910		....	
Population (1910)	6421		7301		6740		1933		3237		....	
Population served	2600		3800		4500		....		....		....	
Sewage, gal. per cap. daily	41		28		60		36		9		80	
Sewage, gal. per cap., connected	102		53		90		94		60		....	
	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.
Nitrogen as:												
Free ammonia[1]	19.4	7.5	28.2	5.7	41.9	14.3	12.4	4.2	34.1	5.0	27.2	7.3
Albuminoid ammonia	11.5	4.4	10.0	2.0	8.4	2.9	3.7	0.6	5.4	0.9	7.8	2.2
Organic nitrogen	27.5	10.6	23.5	4.7	19.3	6.6	7.1	....	12.6	....	18.0	7.3
Oxygen consumed[2]	117.0	45.0	79.0	15.8	62.7	21.4	26.8	3.4	72.0	11.5	71.0	19.4
Chlorine	34.5	13.3	70.0	14.0	62.0	21.1	18.3	4.6	52.8	12.0	47.0	13.0
Solids:												
Total	972.0	376.0	742.8	149.0	489.4	167.0	298.0	84.0	513.2	96.0	603.0	174.0
Volatile	703.0	271.0	488.8	98.0	299.8	102.0	177.2	31.0	298.4	50.0	393.0	110.0
Fixed	269.0	105.0	254.0	51.0	189.6	65.0	120.8	53.0	214.8	46.0	210.0	64.0
Suspended solids:												
Total	767.0	296.0	506.3	101.0	167.5	57.0	99.3	14.0	169.0	32.0	342.0	100.0
Volatile	614.3	237.0	348.4	70.0	127.3	43.0	84.3	11.0	125.2	27.0	260.0	78.0
Fixed	152.7	59.0	157.9	31.0	40.2	14.0	15.0	3.0	43.8	5.0	82.0	22.0
Dissolved solids:												
Total	205.0	79.0	236.5	48.0	321.9	110.0	198.7	69.0	344.2	64.0	261.0	74.2
Volatile	88.7	34.0	140.4	28.0	172.5	59.0	92.9	20.0	173.2	23.0	133.0	32.0
Fixed	116.3	45.0	96.1	20.0	149.4	51.0	105.8	49.0	171.0	41.0	128.0	42.0

[1] Figures given as "free ammonia" have been changed to read "nitrogen as free ammonia." [2] Boiled 5 minutes. Analyses from State Board of Health by courtesy of X. H. Goodnough, Chief Eng. The population contributing sewage is estimated by multiplying the number of house connections by a factor obtained in part by examining the 1903 report of the State Board of Health, and computing the number of persons per connection as there given for the various places. In general, the factor applied was about 7. The total quantity of sewage was obtained from the records of the Board.

TABLE 47.—ANALYSES OF SEWAGE FROM SMALL MASSACHUSETTS TOWNS HAVING SOME MANUFACTURING WASTES

(Parts per million, p.p.m.; grams per capita daily, g.p.c.)

	Framingham[3]		Hudson[3]		Marlboro[3]		Natick[3]		Pittsfield[3]		Westboro[3]		Average	
Date of analyses	1910		1910		1910		1910		1910		1910		.....	
Population (1910)	12,948		6,743		14,579		9,866		32,121		5,446		.....	
Population served	8,000		3,400		11,500		6,000		23,000		3,700		.....	
Sewage, gal. per cap. daily	38		31		38		55		57		48		.....	
Sewage, gal. per cap., connected	61		62		50		90		80		70		69	
	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.pm.	g.p.c.
Nitrogen as:														
Free ammonia[1]	52.7	12.1	45.4	10.6	62.2	11.7	32.0	10.9	19.1	5.8	21.8	5.7	38.9	9.5
Albuminoid ammonia	17.9	4.1	16.8	4.0	11.5	2.2	5.3	1.8	4.4	1.3	11.8	3.2	11.3	2.8
Organic nitrogen	34.7	8.0	39.5	9.3	26.1	4.9	12.0	4.1	9.7	2.9	20.6	5.5	23.8	5.8
Oxygen consumed[2]	129.9	30.0	115.0	27.0	89.8	17.0	47.0	16.0	41.8	12.6	220.0	58.3	107.0	27.0
Chlorine	107.4	24.8	829.7[4]	195.0[4]	113.3	21.4	88.1	30.0	43.8	13.3	62.9	16.6	83.0	21.0
Solids:														
Total	1134.0	262.0	2164.0[4]	509.0[4]	808.0	153.0	550.0	187.0	399.0	121.0	759.0	201.0	730.0	185.0
Volatile	724.0	167.0	649.0[4]	152.0[4]	472.0	89.0	264.0	90.0	202.0	61.0	578.0	153.0	448.0	112.0
Fixed	410.0	95.0	1515.0[4]	357.0[4]	336.0	64.0	286.0	97.0	197.0	60.0	181.0	48.0	282.0	73.0
Suspended solids:														
Total	508.0	117.0	357.0[4]	84.0[4]	323.0	61.0	132.0	45.0	97.0	29.0	152.0	40.0	242.0	58.0
Volatile	423.0	97.0	267.0[4]	63.0[4]	271.0	51.0	106.0	36.0	84.0	24.0	133.0	35.0	203.0	49.0
Fixed	85.0	20.0	90.0[4]	21.0[4]	52.0	10.0	26.0	9.0	15.0	5.0	19.0	5.0	39.0	9.0
Dissolved solids:														
Total	626.0	144.0	1807.0[4]	425.0[4]	485.0	92.0	418.0	143.0	302.0	91.0	607.0	161.0	488.0	126.0
Volatile	301.0	69.0	382.0[4]	90.0[4]	201.0	38.0	158.0	54.0	118.0	35.0	445.0	118.0	245.0	63.0
Fixed	325.0	75.0	1425.0[4]	335.0[4]	284.0	54.0	260.0	89.0	184.0	56.0	162.0	43.0	243.0	63.0

[1] Figures reported as "free ammonia" have been changed to read "nitrogen as free ammonia." [2] Boiled 5 minutes. [3] The population contributing sewage is estimated by multiplying the number of house connections by a factor obtained, in part, by examining the 1903 report of the Mass. State Board of Health and computing the number of persons per connection as there given for the various places. In general, the factor applied was about 7; the total quantity of sewage was taken from the records of the State Board of Health. [4] Not included in average. Analyses from State Board of Health, by courtesy of X. H. Goodnough, Chief Eng.

Wire Mill and Galvanizing Works Wastes.—Very large quantities of acid, usually sulphuric, are used for pickling or cleaning wire and sheet iron or steel before galvanizing and tinning. The waste liquors contain salts of iron and spent acid, and are often utilized to some extent in making copperas and other by-products. In other cases the liquors are discharged into municipal sewers or into streams. They tend to disinfect the sewage and interfere to some extent with biological purification. This is well illustrated by the sewage of Worcester, Mass., which contains an average of 200 p.p.m. of free sulphuric acid and sulphate of iron. It can be successfully purified by intermittent filtration or by trickling filters, but nitrification is usually low because of the acid character of the sewage. As explained in Chapter III, septic decomposition transforms soluble sulphate of iron into insoluble sulphide of iron, which may be precipitated in suitable tanks. Treatment in trickling filters removes large quantities of iron by oxidation to ferric hydrate; a similar oxidation, although much less in extent, occurs in sand filters.

Acid liquors are usually discharged periodically by emptying the pickling vats. The unfavorable effect of these liquors upon the composition of sewage may be materially reduced by storing them at the mills temporarily and discharging them uniformly or in proportion to the flow and strength of the sewage throughout the day. Where chemical precipitation is the method of purification, the iron in these liquors may be of considerable value as a precipitant when treated with lime, as at Worcester.

NOTES ON TABLES 45 TO 48, INCLUSIVE

Chicago, Ill.—The results of analyses of Chicago sewage are those obtained under the direction of Langdon Pearse, Division Engineer, the Sanitary District of Chicago, at the Sewage Experiment Station, 39th Street. The sewage was pumped from the upper side of screens at the mouth of an intercepting sewer (16 ft. in diameter at the lower end), which drains approximately 22.4 square miles and receives the sewage from about 270,000 persons. The average flow was equivalent to about 289 gal. per capita per day. Tributary sewers receive storm water. Sewage pumped to Testing Station by 2½-in. centrifugal pumps, with suction pipes and force main 5 in. diameter. Results shown in Table 45 are averages of weekly samples taken during 1909 and 1910 at hourly intervals, 24 hours per day, 7 days per week. General data and analyses received through courtesy of Pearse. (See also "Sewage Testing Station of Sanitary District of Chicago," by Langdon Pearse, *Engineering News*, vol. lxiii, page 367, 1910.)

Philadelphia, Pa.—The first Philadelphia sewage given in Table 45 was that used at the Spring Garden Testing Station; area of district tributary, 4500 acres; built-up section, 1500 acres; population not estimated; sewage estimated 40 per cent. of domestic origin, considerable ground water; 8 industries contributing dye wastes, 11 strong acids, 8 alkalies, 5 weak acids, 2 soaps, 9 wool washings, 2 yeast and hops; separate sewers; period covered, April, 1909, to April, 1910, inclusive; 75 cc. samples taken every ½ hour, generally for 24 hours 7 days per week; samples taken from force main after passing 12 × 8 × 12-in. pump with 6-in. suction and 4-in. force main, 413 ft. long.

The second Philadelphia analysis in Table 45 represents an estimate (Report by George S. Webster, Chief Engineer, and W. L. Stevenson, Assistant Engineer, upon "Collection, Purification and Disposal of Philadelphia Sewage," 1910, page 200), based on analyses of sewage from 12 drainage areas. These samples were taken in the daytime simultaneously

with gagings of flow. In basing estimates upon these analyses, due weight was given to the time required for the sewage to reach the treatment works. This results in flattening curves of composition and flow.

Boston, Mass.—The results of analyses of Boston sewage, Table 45, are from "Investigations of the Purification of Boston Sewage in Septic Tanks and Trickling Filters" (1905–1907), by C. E. A. Winslow and Earle B. Phelps, *Technology Quarterly*, vol. xx, page 410. They were obtained at the Sanitary Research Laboratory and Sewage Experiment Station of the Massachusetts Institute of Technology. The sewage was taken from an intercepting sewer 9 ft. in diameter serving the City of Boston, and was collected by combined sewers from a contributing population of 250,000; average flow, 200 gal. per capita per day; sewage pumped by 4 × 6-in. duplex pump to small grit chamber, where samples were taken every 3 hours day and night, at once chloroformed and combined to make composite sample for analysis at end of each week. Period covered by analyses, October, 1905, to June, 1907. When considering the results of these analyses computed in terms of grams per capita, it must be remembered that the sewage is admitted to the intercepter through automatically controlled gates and that when there is run-off from storms very large quantities of sewage and storm water pass directly to the harbor, thus by-passing much sewage matter and street detritus which is unaccounted for in the analyses and computations.

Columbus, Ohio.—The results of analyses of the Columbus sewage, Table 45, are from "Report on Sewage Purification, Columbus, Ohio," by George A. Johnson, 1905. The sewage was pumped from an intercepting sewer at a point about 1.75 miles from center of city. Area of main sewerage district, 6790 acres, practically all built up; tributary population, 75,000; population of district, 100,000; collecting system and intercepters, 112 miles, exclusive of house connections; overflows provided at all junctions of trunk sewers with intercepter, permitting excess storm flows to pass to river. At times clogging occurred at these junctions, permitting part of ordinary flow to pass directly to river. This condition and fact that storm flows in excess of capacity of intercepter passed directly to river, should be taken into account when considering composition of sewage computed in terms of grams per capita. Considerable quantities of industrial wastes were admitted to sewers, including those from dye houses, tanneries, breweries and iron works. Samples taken after screening at half-hour intervals throughout 24 hours for 7 consecutive days. On eighth day, no sample taken, new schedule begun on ninth day and repeated. Half-hourly portions combined to make 24-hour samples which were analysed. Period of tests, August 16, 1904, to June 30, 1905 (319 days) during which 282 samples were analysed. Average composition weighted proportionately to number of samples analysed for each month.

Milwaukee, Wis.—The analysis of Milwaukee sewage, Table 45, is hypothetical, being an estimate by the Milwaukee Sewage Disposal Commission, 1910, based upon analyses of domestic sewage, sewage containing industrial wastes, and industrial wastes. The industrial wastes considered in compiling this hypothetical analysis are those from packing houses, breweries and asbestos works.

Providence, R. I.—The analysis of Providence sewage, Table 45, is from the report of the City Engineer, 1909, page 50. Providence is served chiefly by combined sewers, trunk sewer connections being provided with storm overflows to permit excess of sewage and storm water to flow to the river in time of storm. Most sewage is screened and pumped before reaching the sampling station. Textile mills employing 11,600 hands, discharge 5,000,000 gal. industrial wastes daily into the Valley Street sewer, while there are various other mills at other parts of the city producing wastes of a similar character. One million pounds of wool have been scoured in 1 week. Equal samples are taken hourly while pumps are running, about 15 hours per day. These portions make up a 24-hour composite sample each day, from which a portion proportional to the flow for the day is taken, to make up a 7-day sample, which is analysed.

Worcester, Mass.—The results of analyses of Worcester sewage, Table 46, are from the annual report of the Superintendent of Sewers, 1910, page 70. The sewage flowing during a portion of the day is treated by chemical precipitation, that flowing during the remainder of the day being passed through intermittent sand filters. Samples of sewage to be chemically treated are taken every half hour, in proportion to the quantity flowing. These portions are combined, making 1 sample for analysis combining portions taken during 7 days. Equal samples are taken half hourly of the sewage going to the filter beds after it has passed through the grit chambers. These portions are combined into 1 weekly sam-

ple which is analyzed. The analyses of the sewage filtered and the sewage treated chemically are combined in accordance with the quantity treated in each way. These combined results are given in Table 46.

A portion of the city is provided with a separate system of sewers, the remainder being served by combined sewers. Combined trunk sewers are connected to the intercepters by means of regulators consisting essentially of gates which automatically regulate the quantity admitted to the intercepters during times of storm, the surplus being discharged into Mill Brook. In considering the composition of this sewage when expressed in grams per capita, consideration should be given to the fact that large quantities of sewage and storm water escape into the river and are, therefore, not included in the analyses and computations.

Worcester is an industrial center, large quantities of liquid wastes being produced by packing houses, breweries, woolen mills, bleach and dye works, carpet mills, wire-drawing and galvanizing works and tanneries. The quantity of acid wastes from the wire mills is so great that they materially influence the character of the sewage. The sewage received at the treatment plant is measured and the population given is that of 1910 Census.

Gloversville, N. Y.—The analysis of the Gloversville sewage, Table 46, was based upon results obtained from the operation of a sewage testing station. (Report on Sewage Purification Experiments and Sewage Disposal, Harrison P. Eddy and Morrell Vrooman, 1909, page 59.)

Gloversville is provided with a separate system of sewers and had, in 1908, 22 fine leather tanneries, 1 knitting mill and 1 silk mill that discharged relatively large quantities of industrial wastes into city sewers. This had a material effect upon the physical and chemical character of the sewage. The former was manifested by pronounced colors from spent dye liquors and by hair and fleshings, while the latter was indicated by high values for nitrogen, oxygen consumed, alkalinity and fats. Period covered by analyses, May, 1908, to June, 1909.

The industrial plants connected with the city sewers had settling tanks to remove the heavier suspended matter. Samples of sewage were taken from the trunk sewer every hour and mixed together at the end of 24 hours in the proportion of 2 parts day sewage (7 a.m. to 6 p.m.) to 1 part night sewage (7 p.m. to 6 a.m.). This ratio was based upon sewer gagings which showed the day flow to be approximately twice the night flow. The averages were based upon the analyses of 400 composite samples. The analyses were made by Harry B. Hommon, chemist in charge of the sewage purification experiments.

Waterbury, Conn.—The analysis of Waterbury sewage in Table 46 is from an article on "Waterbury Sewage and Its Septic Action," by William Gavin Taylor, *Engineering News*, June 3, 1909, page 596. Period covered by analysis, November, 1905, to November, 1906; analysis given is average of 314 daily composite samples collected in half-hour portions after coarse screening. Combined sewers. The industries of Waterbury are largely confined to manufacture of clocks, watches and brass goods. Sewage substantially free from refractory trade wastes and is alkaline in spite of considerable acid pickle.

Massachusetts Cities.—The analyses of sewage from Massachusetts cities and towns, except Boston and Worcester, are from records of the State Board of Health, furnished by courtesy of X. H Goodnough, Chief Eng.

At Brockton samples are taken throughout 24 hours of 1 day each month, the composite sample being analyzed. In the other cases single samples, usually taken in the daytime once a month, were analyzed. While probably none of these samples, except that of Brockton, was representative of the 24 hours' flow, it is probable that the averages covering the samples for the entire year represent the quality of the sewage fairly well.

Brockton is essentially a shoe manufacturing city and the quantity of liquid wastes reaching the sewers is relatively small.

At Clinton there are carpet mills from which liquid wastes, including wool scouring liquors, are discharged into the sewers.

At Framingham liquid wastes from dyeing and bleaching establishments are discharged into the sewers.

At Hudson the sewage is materially affected by a relatively large quantity of tannery wastes.

At Pittsfield the wastes from woolen mills reach the sewers.

At Marlboro and Natick there is very little liquid industrial waste as the principal industries are the manufacture of shoes. The towns included in Table 48 are all rural residential communities.

TABLE 49.—ANALYSES OF SEWAGE OF ENGLISH CITIES, CONTAINING TRADE WASTES

(Parts per million, p.p.m.; grams per capita daily, g.p.c.; partly from "Sewage Disposal," by Kinnicutt, Winslow and Pratt, page 8)

	Birmingham[1]		Bradford[2]		Leeds[3]		Leicester[4]		Manchester[5]		Sheffield[6]		Average	
Population connected with sewers	900,000		240,000		454,450		235,000		575,000		400,000		......	
Dry-weather flow, U. S. gal., daily	32,544,000		15,600,000		20,648,400		11,551,244		30,360,000		19,200,000		......	
Dry-weather flow, gal. per cap.	36		65		45.5		47.4		52.8		48		49.1	
	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.
Nitrogen as:														
Free ammonia	41.6	5.68	47.8	11.7	22.8	3.9	45.1	8.3	29.3	5.8	32.2	5.8	36.5	6.9
Albuminoid ammonia	16.2	2.21	32.8	8.2	7.9	1.4	15.6	2.8	7.0	1.4	9.0	1.6	14.8	2.9
Oxygen consumed in 4 hr.:														
Unfiltered	259.2	35.3	202.3	49.8	99.0	17.0	125.2	22.1	113.6	22.7	79.6	14.5	146.5[8]	26.9[9]
Filtered	138.9	18.9	......	......	......	......	......	......	......	......	......	......	......	......
Chlorine	205.0	28.0	149.0	36.7	165.0	28.4	127.2	23.0	200.0	40.0	124.0	22.6	162.0	30.0
Solids:														
Total	1947.0	266.0	2650.0	652.0	1843.0	317.0	1805.0	326.0	1235.0	247.0	......	......	1896.0	362.0
Volatile	......	......	1774.0	437.0	......	......	......	......	592.0	118.0	......	......	......	......
Fixed	......	......	876.0	215.0	......	......	......	......	643.0	129.0	......	......	......	......
Suspended solids:														
Total	718.0	98.0	840.0	207.0	775.0	133.0	680.0	123.0	327.0	65.0	418.0[7]	76.0	668.0	126.0
Volatile	......	......	264.0	65.0	......	......	......	......	163.0	32.0	171.0	31.0	......	......
Fixed	......	......	576.0	142.0	......	......	......	......	164.0	33.0	247.0	45.0	......	......
Dissolved solids:														
Total	1229.0	168.0	1810.0	445.0	1069.0	184.0	1125.0	203.0	908.0	182.0	......	......	1228.0	236.0
Volatile	......	......	1510.0	372.0	......	......	869.0	157.0	429.0	86.0	......	......	......	......
Fixed	......	......	300.0	73.0	......	......	256.0	46.0	479.0	96.0	......	......	......	......

[1] From "Birmingham Sewage Disposal Works," Watson, *Proc. Inst. C. E.*, 1910; average for 1901–5. [2] General average; communicated by Garfield, August, 1909. [3] Average of 23 analyses; communicated by Hart, July, 1909. [4] Average of 50 analyses; communicated by Mawbey, September, 1909. [5] Average for 1905; communicated by Fowler, August, 1909. [6] Average of 130 analyses; communicated by Wike, August, 1909. [7] Not included in average. [8] Equivalent to about 266 when boiled 5 minutes. [9] Equivalent to about 49 when boiled 5 minutes.

TABLE 50.—ANALYSES OF SEWAGE FROM COMMUNITIES IN THE EMSCHER DISTRICT, GERMANY, SEPTEMBER, 1912.—(*Continued*)

	Recklinghausen		Schwerin		Sodingen		Teutoburgia		Suderwich		Westhausen		Average of 13 places	
Population	30,000		3,100		7,000		3,000		4,000		2,000		......	
Sewage, gal. per cap.	79		17		75		53		60		69		......	
	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.
Nitrogen as:														
Free ammonia	25.2	7.6	44.4	2.8	29.8	8.3	19.8	4.0	16.5	3.8	12.2	3.2	33.7	7.5
Organic nitrogen	11.7	3.5	26.9	1.7	12.1	3.4	15.2	3.0	11.1	2.6	7.7	2.0	16.9	3.6
Total	36.9	11.1	71.3	4.6	41.9	11.7	35.0	7.0	27.6	6.3	20.9	5.4	50.6	11.1
Chlorine	199.0	60.0	217.0	14.0	490.0[2]	137.0	172.0	34.0	95.0	22.0	1296.0[3]	337.0	175.0[3]	......
Solids:														
Total	1375.0	412.0	1615.0	104.0	1959.0	549.0	1339.0	268.0	1735.0	399.0	3666.0	954.0	2044.0	550.0
Volatile	475.0	142.0	608.0	39.0	403.0	112.0	437.0	87.0	693.0	159.0	779.0	203.0	623.0	151.0
Fixed	900.0	270.0	1007.0	65.0	1556.0	437.0	902.0	181.0	1042.0	240.0	2887.0	751.0	1421.0	399.0
Suspended solids:														
Total	440.0	132.0	504.0	33.0	225.0	63.0	491.0	98.0	694.0	160.0	456.0	119.0	601.0	153.0[4]
Volatile	259.0	78.0	297.0	19.0	114.0	31.0	237.0	47.0	371.0	85.0	234.0	61.0	332.0	82.0
Fixed	181.0	54.0	207.0	14.0	111.0	32.0	254.0	51.0	323.0	75.0	222.0	58.0	269.0	71.0
Dissolved solids:														
Total	935.0[1]	280.0	1111.0[1]	71.0	1734.0[1]	486.0	848.0[1]	170.0	1041.0[1]	239.0	3210.0[1]	835.0	1443.0	397.0
Volatile	216.0	64.0	311.0	20.0	289.0	81.0	200.0	40.0	322.0	74.0	545.0	142.0	291.0	69.0
Fixed	719.0	216.0	800.0	51.0	1445.0	405.0	648.0	130.0	719.0	165.0	2665.0	693.0	1152.0	328.0

[1] Dissolved matter in effluent from Imhoff tanks; assumed equal to that in sewage. [2] High chlorine due to waste water from mines. [3] Excluding those affected by mine waste water. [4] This figure is higher than those given in Frühling's "Entwässerung der Städte," which are: Allenstein, 35; Berlin, 76; Breslau, 50; Charlottenburg, 44.

TABLE 50.—ANALYSES OF SEWAGE FROM COMMUNITIES IN THE EMSCHER DISTRICT, GERMANY, SEPTEMBER, 1912

(Parts per million, p.p.m.; grams per capita daily, g.p.c.; analytical results furnished by Dr. D. W. Bach, chemist of the Emschergenossenschaft)

	Aplerbeck		Bochum		Essen Fronhausen		Essen Nord		Essen Nordwest		Holzwickede		Lütgen-dortmund	
Population	3,000		145,000		40,000		180,000		92,000		3,500		4,000	
Sewage, gal. per cap.	79		91		36		102		138		51		25	
	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.	p.p.m.	g.p.c.
Nitrogen as:														
Free ammonia	32.0	9.6	23.5	8.0	82.8	11.6	28.2	11.0	32.9	17.1	18.0	3.4	72.7	6.9
Organic nitrogen	21.8	6.5	9.2	3.1	25.1	3.5	13.1	5.1	11.3	5.9	15.5	2.9	39.0	3.7
Total	53.8	16.1	32.7	11.1	107.9	15.1	41.3	16.1	44.2	23.0	34.6	6.6	111.7	10.6
Chlorine	120.0	36.0	858.0[2]	292.0	248.0	35.0	482.0[2]	188.0	814.0[2]	423.0	132.0	23.0	219.0	21.0
Solids:														
Total	1444.0	433.0	2678.0	911.0	2188.0	306.0	2126.0	829.0	2915.0	1516.0	1334.0	253.0	2194.0	208.0
Volatile	524.0	157.0	512.0	174.0	1049.0	147.0	495.0	193.0	708.0	368.0	473.0	90.0	939.0	89.0
Fixed	920.0	276.0	2166.0	737.0	1139.0	159.0	1631.0	636.0	2207.0	1148.0	861.0	163.0	1255.0	119.0
Suspended solids:														
Total	546.0	164.0	458.0	156.0	925.0	129.0	751.0	293.0	898.0	467.0	400.0	76.0	1024.0	97.0
Volatile	340.0	102.0	254.0	86.0	594.0	83.0	388.0	151.0	433.0	225.0	194.0	37.0	599.0	57.0
Fixed	206.0	62.0	204.0	70.0	331.0	46.0	363.0	142.0	465.0	242.0	206.0	39.0	425.0	40.0
Dissolved solids:														
Total	898.0[1]	269.0	2220.0[1]	755.0	1263.0[1]	177.0	1375.0[1]	536.0	2017.0[1]	1049.0	934.0	177.0	1170.0[1]	111.0
Volatile	184.0	55.0	258.0	88.0	455.0	64.0	107.0	42.0	275.0	143.0	279.0	53.0	340.0	32.0
Fixed	714.0	214.0	1692.0	667.0	808.0	113.0	1268.0	494.0	1742.0	906.0	655.0	124.0	830.0	79.0

TABLE 51.—ANALYSES OF SEWAGE FROM GERMAN COMMUNITIES

(Parts per million)

(Analyses collected by Dr. D. W. Bach, Chemist of the Emschergenossenschaft; abbreviated dates are given in the order of month, day, year)

City	Bremen, "Hemmgraben"	Bremen, "Waller"	Bremen, "Weser River District"	Breslau, without w.c. connections	Breslau, with w.c. connections	Cologne
Date of collection	1903–1906	8/11/04 and 11/9/05	4/26/04 and 1/5/05	1890–1898	....	1901–1902
Population	258,670 (1912)	....	....	....	....	500,000 (1910)
Nitrogen as:						
Free ammonia	Total nitrogen 21–43	6.7	15.0	24.7	74	30.0
Alb. ammonia		Total nitrogen 11.4	Total nitrogen 26.8	....	....	6.0
Organic				2.6	18	20.0
Nitrites	....			....	....	0–4
Nitrates	....	....	....	....	....	0–12
Oxygen consumed[1]	25–81	23.5	100.6	....	125	68.7
Chlorine	160–230	99.3	269.8	79.0	....	140.0
Solid matter:						
Total	....	651.0	1121.0	940.0	1178	1195.0
Volatile	....	181.0	310.0	....	443	444.0
Fixed	....	470.0	811.0	....	735	751.0
Suspended matter:						
Total	100–350	94.0	204.0	211.0	405	303.0
Volatile	60[2]	49.0	112.0	....	200	215.0
Fixed	40[2]	45.0	92.0	....	205	88.0
Dissolved matter:						
Total	720–950	557.0	917.0	729.0	773	892.0
Volatile	125–225	132.0	198.0	334.0	243	229.0
Fixed	....	425.0	719.0	395.0	530	663.0

[1] Ten-minute Kubel test. [2] Percentage of total.

Bremen figures from Strassenbau-inspection der Stadt Bremen, Breslau figures from Stadtbauamt, and Cologne figures from Tiefbauamt.

TABLE 51.—ANALYSES OF SEWAGE FROM GERMAN COMMUNITIES.—(*Continued*)

City	Cassel	Danzig	Dort-mund	Düssel-dorf	Frankfort a. Main	Görlitz	Hagen
Date of collection	1910	before 1899	before 1899	1/2/01–1/3/01	1907	1912	7/31/11–9/2/12
Population	160,000	....	213,000	400,000	410,000 connected (1912)	86,000	93,000
Nitrogen as:							
Free ammonia	41.8	53.2	27	22.1	42	80	11.5
Alb. ammonia	....	....	....	....	9	....	....
Organic	18.0	11.6	26	0.7	11	40	44.2
Nitrites	....	....	....	....	....	....	....
Nitrates	....	....	....	....	0–2	....	....
Oxygen consumed	146.4	....	115	119.4	89	184	54.1
Chlorine	66.8	70.0	135	....	140	197	129.8
Solid matter:							
Total	....	1265.0	1396	1103.0	1152	1960	1000.0
Volatile	....	517.0	528	361.0	449	1172	403.0
Fixed	....	748.0	868	742.0	703	788	597.0
Suspended matter:							
Total	230–330	582.0	430	252.0	411	868	313.0
Volatile	67–224	356.0	244	160.0	241	691	173.0
Fixed	77–156	226.0	186	92.0	170	177	140.0
Dissolved matter:							
Total	245–500	683.0	966	851.0	741	1092	687.0
Volatile	40–290	161.0	284	201.0	208	481	230.0
Fixed	200–310	522.0	682	650.0	533	611	457.0

Cassel figures from Städtisches Bauamt, Dantzig figures from Stadtbauamt, Dortmund figures from Städtisches Tiefbauamt, Düsseldorf figures from Tiefbauamt II (Kanalisationswerke), Frankfort figures from Tiefbauamt (Klärbecken-Betriebs-inspection), Görlitz figures from Tiefbauamt, and Hagen figures from Stadtgemeinde.

TABLE 51.—ANALYSES OF SEWAGE FROM GERMAN COMMUNITIES.—(*Continued*)

City	Halle a. Salle	Halle a. Salle without w.c. connections	Hannover	Mannheim (considerable mill trade wastes)	Ottensen, without w.c. connections
Date of collection	before 1899	....	June, 1897	1906–1912	before 1899
Population	181,000	....	270,000 connected (1910)	180,000 connected (1910)	....
Nitrogen as:					
Free ammonia	89	67.8	....	30	47.6
Alb. ammonia	....	....	....	....	....
Organic	59	21.3	....	8–10	20.7
Total	....	....	76	....	....
Nitrites	....	....	....	....	....
Nitrates	....	....	....	....	....
Oxygen consumed	99	21.6	47	20–75	115.0
Chlorine	715	209.0	171	90–170	628.0
Solid matter:					
Total	3388	2458.0	1315	....	2478.0
Volatile	995	752.0	405	....	809.0
Fixed	2393	1706.0	910	....	1669.0
Suspended matter:					
Total	594	825.0	338	300–350	661.0
Volatile	405	423.0	219	50–70[1]	442.0
Fixed	189	402.0	119	30–50[1]	219.0
Dissolved matter:					
Total	2794	1633.0	977	500	1817.0
Volatile	590	329.0	186	....	367.0
Fixed	2204	1304.0	791	....	1450.0

[1] Percentages of total.

Halle figures from Stadtbauamt, Hannover figures from Magistrat, Mannheim figures from Städtisches Tiefbauamt, Ottensen figures from König's "Verunreinigung der Gewasser."

TABLE 51.—ANALYSES OF SEWAGE FROM GERMAN COMMUNITIES.—(*Continued*)

City	Pforsheim (considerable iron trade wastes)	Strassburg (experimental sewage plant)	Stuttgart (experimental sewage plant)	Unna
Date of collection	1911–1912	1911–1912	1905	1912
Population	69,084	178,290 (1910)	300,000 (1910)	18,000
Nitrogen as:				
Free ammonia	16.2		.....	44.3
Alb. ammonia	.....	.....	.....	7.3
Organic	32.0	16.1	.....	14.2
Total	.....	.....	.....	.....
Nitrites	.....	.....	.....	0.3
Nitrates	.....	.....	.....	32.0
Oxygen consumed	30.2	42.8	48	.....
Chlorine	86.0	50–80	.....	163.2
Solid matter:				
Total	775.0	.....	1888	1531.0
Volatile	352.0	.....	760	775.0
Fixed	423.0	.....	1128	756.0
Suspended matter:				
Total	220.0	500–850	450	597.0
Volatile	110.0	.....	208	479.0
Fixed	110.0	.....	242	118.0
Dissolved matter:				
Total	555.0	.....	1438	934.0
Volatile	242.0	.....	552	296.0
Fixed	313.0	.....	886	638.0

Pforsheim figures from the Städtisches Tiefbauamt, Strassburg figures from the Kaiserliches Ministerium für Elsass-Lothringen, Stuttgart figures from the Städtisches Tiefbauamt, and Unna figures from the Stadtbauamt.

TABLE 52.—COMPARATIVE AVERAGE SEWAGE ANALYSES

(Compiled from Tables 45 to 51)

Constituents	Parts per million						Grams per capita					
	Large American cities (combined sewers)	American mfg. cities	Small American mfg. cities	American residential and rural communities	Large English mfg. cities	German communities of the Emscher district	Large American cities	American mfg. cities	Small American mfg. cities	American residential and rural communities	Large English mfg. cities	German communities of the Emscher district
Sewage flow, gal. per cap. per day.	178.0	95.0	69.0	80.0	49.0	....	....	....	....	....	....	....
Nitrogen as:												
Free ammonia	10.6	26.5	38.9	27.2	35.5	33.7	7.8	7.8	9.5	7.3	6.9	7.5
Albuminoid ammonia	7.0	11.9	11.3	7.8	14.7	....	....	2.7	2.8	2.2	2.9	3.6
Organic nitrogen	8.0	24.1	23.8	18.0	....	16.9	6.4	8.0	5.8	7.3	....	11.1[1]
Nitrites	0.11	0.26	....	....	....	....	0.05	....	....	....	....	....
Nitrates	0.44	1.19	....	....	....	....	0.21	....	....	....	....	....
Oxygen consumed	59.0	133.0	107.0	71.0	266.0	....	35.6	48.0	27.0	19.0	49.0	....
Chlorine	48.0	109.0	83.0	47.0	162.0	209.0	37.0	45.0	21.0	13.0	30.0	....
Alkalinity	153.6	129.0	....	....	....	....	161.0	....	....	....	....	....
Solids:												
Total	1355.0	1058.0	730.0	603.0	1896.0	2044.0	567.0	266.0	185.0	174.0	362.0	550.0
Volatile	453.0	635.0	448.0	393.0	....	623.0	185.0	149.0	112.0	110.0	....	151.0
Fixed	903.0	423.0	282.0	210.0	....	1421.0	382.0	117.0	73.0	64.0	....	399.0
Suspended solids:												
Total	303.0	450.0	242.0	342.0	668.0	601.0	126.0	97.0	58.0	100.0	126.0	153.0
Volatile	211.0	365.0	203.0	260.0	....	332.0	86.0	74.0	49.0	78.0	....	82.0
Fixed	92.0	85.0	39.0	82.0	....	269.0	40.0	23.0	9.0	22.0	....	71.0
Dissolved solids:												
Total	1052.0	608.0	488.0	261.0	1228.0	1443.0	441.0	169.0	126.0	74.0	236.0	397.0
Volatile	242.0	270.0	245.0	133.0	....	291.0	99.0	75.0	63.0	32.0	....	69.0
Fixed	811.0	338.0	243.0	128.0	....	1152.0	342.0	94.0	63.0	42.0	....	328.0
Fats	25.0	37.0	....	....	....	....	25.0	....	....	....	....	....

[1] Total nitrogen.

Note.—German figures are based on flow per capita of total population, therefore probably somewhat lower than strictly comparable figures.

TABLE 53.—ANALYSES OF SEWAGE OF FRENCH COMMUNITIES

(Parts per million)

	Fougères, Ille et Vilaine (1)	Ivry et Vitry, Seine (2)	Paris, Seine (3)	La Madeleine-lez-Lille (Nord.) (4)	Average
Population	....	44,000	2,763,000	....	....
Date of analyses	July 5, 1909	1909	1909	1905–11–12	....
Nitrogen as					
Free ammonia	34	20.9	27.2	16.1	24.6
Organic	20	....	4.5	18.6	14.4
Nitrites	....	....	....	....	....
Nitrates	....	....	....	....	....
Oxygen consumed	83	33.0	32.9	98.5	61.9
Chlorine	202	64.0	62.0	236.0	141.0
Alkalinity	290	....	....	494.0	392.0
Solids:					
Total	896	838.0	....	2442.0	1392.0[1]
Volatile	213	....	....	1030.0	621.5
Fixed	683	....	....	1412.0	1047.5
Suspended solids:					
Total	76	104.0*	....	736.6	305.5[2]
Volatile	28	....	....	347.0	187.5
Fixed	48	....	....	389.7	218.8
Dissolved solids:					
Total	820	....	....	1211.0	1015.5
Volatile	185	....	....	464.0	324.5
Fixed	635	....	....	747.0	691.0

Analyses from the following sources through courtesy of M. Rolants, Pasteur Institute of Lille; (1) Pasteur Institute of Lille; (2) Lab. of Hygiene of Paris (aver. of 7 analyses); * Lab. of Sewerage Works of Ivry and Vitry. (3) Lab. of Hygiene, Paris. (4) Pasteur Institute of Lille.

Notes by M. Rolants.—(1) 25,537 inhabitants; a manufacturing town discharging little industrial waste. Sewage of combined system is composed probably wholly of household sewage, except night soil, and rain-water (the last wholly lacking or present in very small quantities in July). The samples were taken hour by hour at the discharge end of the sewer. I cannot affirm that the figure given for suspended matter, represents the daily average. (2) 44,000 inhabitants. Sewage is composed of rain-water, household and manufacturing wastes. Probably very little night soil. (4) It is not possible to exactly enumerate the population of the territory from which the sewage is received at the experiment station. One can only estimate it at 5000 to 6000. This sewage is composed of rain-water, household and industrial wastes, with almost complete exclusion of night soil. Figures in column 4 are averages of tests covering May, 1905, 1 week in February, 1911, and 7 days each in Dec., 1911, and Feb., May and June, 1912.

[1] Average of columns 1 and 4 is 1669.   [2] Average of columns 1 and 4 is 406.3.

TABLE 54.—RESULTS OF CHEMICAL ANALYSES OF INDUSTRIAL WASTES

(Parts per million)

	Residue on evaporation						Nitrogen as ammonia								Organic nitrogen		
	Total residue			Loss on ignition				Albuminoid									
	Total	Dis-solved	Sus-pended	Total	Dis-solved	Sus-pended	Free	Total	Dis-solved	Chlorine	Oxygen consumed	Sulphur	Fats	Alkalinity	Total	Dis-solved	Acidity
Candle factory wastes[1]	4,108	3,044	1,064	1,872	768	1,104	...	...	...	50	650	482	1,624	...	...	...	...
Soap factory wastes	16,612	9,748	6,864	8,246	2,036	6,210	11.08	...	...	1,594	5,007	808	6,162	...	67.72	...	...
Packing and rendering wastes	4,833	4,202	631	1,854	1,313	541	12.72	...	...	1,212	935	81	346	...	159.78	87.28	292
Glue and fertiliser wastes[1]	3,746	840	2,906	2,008	244	1,764	9.40	...	...	82	485	117	408	...	58.60	2.60	...
Creamery wastes[1]	1,450	...	...	1,307	...	...	1.75	38.1	5.9	...	512	...	...	...	...	...	...
Rubber mill wastes	...	...	1,060	...	...	691	2.1	...	...	596	331	...	...	816	9.72	...	...
Salt factory wastes[1]	...	...	65	...	...	19	2.1	...	...	74,000	64	...	...	66	1.3	...	...
Distillery wastes	6,653	3,920	2,733	5,857	3,204	2,653	5.40	...	...	28	5,070	...	...	...	218.6	106.6	583[3]
Brewery wastes[1]	3,396	3,048	348	2,512	2,250	262	0.30	...	...	...	1,226	...	...	...	39.7	...	...
Yeast factory wastes[1]	31,990	...	...	24,625	...	...	36.1	253.0	234.0	...	8,007	...	...	...	...	...	...
Bleach and dye works (cotton)	3,190	2,540	650	990	560	430	1.95	5.13	3.50	140	350	20[3]	50[3]	...	50.0[3]	38.0[3]	...
Wool scouring	64,300	32,800	31,500	33,600	14,500	19,100	97.0	222.0	120.0	1,845	3,800	160[3]	23,740	...	259.0[3]	195.0[3]	...
Cloth wash, dye, general wool mill wastes.	4,180	2,780	1,400	2,190	1,140	1,050	10.9	22.6	10.0	370	523	37	245	...	50.0[3]	25.0[3]	...
Shoddy mill wastes[1]	717	...	...	369	...	...	6.5	3.5	...	...	37	...	...	480	...	...	...
Paper mill: General wastes	620	260	360	290	150	140	0.67	1.80	...	290	110	30[3]	...	...	...	...	...
Paper mill: Washer wastes	2,800	1,300	1,500	1,100	500	600	1.07	5.25	3.65	47	290	...	...	...	...	...	...
Paper mill: Machine[2] wastes	1,200	350	850	400	100	300	0.14	0.53	0.17	16	70	...	...	...	...	...	...
Paper mill: Sulphite pulp wastes	75,000	...	...	65,000	...	...	...	...	...	...	35,000	3,500	...	...	...	...	...
Paper mill: Straw-board wastes[1]	20,000	10,000	10,000	13,000	8,000	5,000	...	...	...	...	9,000	...	...	...	...	...	...
Tannery wastes	4,000	3,100	900	1,860	1,230	630	23.0	326.0	134.0	1,680	830	200	260	...	80.0	35.0	...

[1] Analyses from one mill only.

[2] These wastes contained 0.09 p.p.m. nitrates.

[3] Based on but one or two analyses, which may not be typical.

CHAPTER VI

THEORIES OF SEWAGE DISPOSAL AND TREATMENT

The most logical method of disposing of sewage, and that first adopted in many communities, is to discharge it into the nearest body of water. When the processes of nature by which the water is kept inoffensive are overworked, the sewage-laden waters become offensive. It is then that the treatment of sewage becomes necessary to assist the forces of nature which had prevented any nuisance, and it is interesting to note that artificial treatment employs agencies very similar to those utilized by nature in carrying out the work thrust upon her by the advent of human activities. The skill of the engineer is shown in selecting that agency or combination of agencies which will produce the desired results most economically.

At the outset it is important to acquire a clear comprehension of the terms used in discussing the work done by the several processes of sewage treatment. Prof. George C. Whipple says:

"Unfortunately the term 'sewage purification' is popularly applied to any one or all of these processes. This has contributed not a little to confusion of ideas. Laymen innocently suppose that when sewage is 'purified' it becomes pure, whereas the sanitary engineer may mean only that it is purer than it was before. How much purer, depends upon the method used. It would be of decided benefit to the cause of sanitation if instead of using the term 'sewage purification' in a false sense, as we often do, we used more definite expressions, saying 'sewage clarification' when we mean the removal of suspended matter, 'oxidation' or 'deodorization' when we mean the removal of putrescibility, and 'sewage disinfection,' 'sewage filtration,' etc., to describe the different parts of the process, leaving the term 'sewage purification' to be used in a generic sense and understanding that it does not necessarily mean complete purification." (*Eng. Record*, Jan. 7, 1911, vol. lxiii, page 20.)

The authors are inclined to go still further and to refer to the work accomplished, so far as possible, by the names of the processes employed, using the terms "screened sewage" or "settled sewage" instead of "clarified sewage." The most important suggestion in Whipple's statement, however, is to avoid giving the impression that sewage is made pure by screening, sedimentation, or even by the most effective filtration ordinarily employed, and, when using the word "puri-

fication," to make certain that it conveys simply the impression that the sewage is deprived of some of its objectionable constituents, although it never becomes pure water. Accordingly we may define "sewage purification" to be the work of a process or group of processes by which sewage is so changed as to render it more like the water of which it is so largely composed. This may be accomplished by the actual removal of a portion of its constituents, by a transformation of organic matter into mineral or more stable organic substances, by destroying bacterial life, or by a combination of two or more of these acts.

It is also of fundamental importance to understand that sewage need not be brought to the same degree of purification under all conditions. Obviously, small quantities of raw sewage[1] may be discharged with impunity into many bodies of water, and in a general way the need of treatment and the degree of purification required increase with the increase of the proportion of sewage to diluting water.

The primary object in studying methods of sewage disposal is to find means of getting rid of sewage without its causing offensive conditions or becoming a menace to health. This must, in every instance, be accomplished ultimately by its discharge, before or after treatment, into some body of water. Three different objects are sought in the artificial treatment of sewage:

1. To prevent waters into which sewage is discharged from becoming offensive to the eye because of floating matter.
2. To prevent such waters from becoming malodorous.
3. To prevent the introduction into the water of germs of disease.

These objects are attained by the following methods of treatment, or combinations of two or more of them:

1. Screening.
   (a) Coarse.
   (b) Fine.
2. Sedimentation.
   (a) Rapid in grit chambers.
   (b) Slow in settling basins.
      Plain.
      With chemical precipitation.
      With septic action in single-story tanks.
      With sludge digestion in 2-story tanks.
      With colloiders.
   (c) With aeration.
      Without the use of organisms or sludge from previously aerated sewage.
      With the use of organisms or sludge from previously aerated sewage.

[1] Meaning sewage as discharged from a system of sewers and not treated in any way.

3. Contact bed treatment.
4. Trickling filter treatment.
5. Intermittent filtration.
6. Broad irrigation.
7. Disinfection.

Prior to 1890, little attention was given to objectionable conditions produced by the discharge of sewage into American rivers. Since that time, however, there has been a tendency toward progressively greater restriction upon the discharge of untreated sewage and toward constantly raising the degree of purification required. While there can be little doubt that in certain cases progress in these directions has exceeded logical and reasonable requirements, it is equally true that over the greater part of this country the requirements are still quite lax and more care in disposing of sewage will be necessary in the future than has been exercised in the past. It is desirable, therefore, when adopting a plan for the disposal of sewage to select for present use such process or processes as may meet the needs of the present and immediate future, but at the same time to plan for more effective means when they shall be required.

COMPOSITION OF SEWAGE

Before taking up the theories involved in common methods of treatment, it is important to form a clear conception of the physical condition of the constituents of sewage. The great variation in composition of sewage has been discussed in detail in Chapter V, from which the hypothetical analysis, page 156, may be taken to represent American sewage of medium strength, such as may be expected in a city of 100,000 to 300,000 population with diversified industries, some of which produce liquid wastes, where a moderate quantity of water is consumed and where most of the sewers receive storm water.

It will be seen in that analysis that the substances, other than water, in sewage are about one-half organic and one-half mineral. About three-eighths of the organic matter consists of nitrogenous substances containing nitrogen, carbon, hydrogen, oxygen, sulphur, phosphorus and other elements. The other five-eighths of the organic matter consists of carbohydrates, fats and other like substances containing carbon, hydrogen and oxygen but no nitrogen. The solids in the three physical forms in the hypothetical sewage of Chapter V, page 156, may be grouped as in Fig. 16. Those suspended solids capable of settling in 2 hours have been designated as settling solids and the remainder as suspended colloidal solids. If the period of sedimentation were longer, the quantity of settling solids would be greater and the quantity of suspended colloidal solids correspondingly less. There are few data upon which to predicate such an estimate as the foregoing, particularly

with reference to the colloidal solids. The schematic arrangement is purely hypothetical, introduced merely to illustrate a classification of the constituents of sewage according to their physical condition in a manner helpful in the study of problems of sewage disposal.

Formerly attention was directed wholly to solids in suspension and in solution, which were separated by filtering the sewage through ordinary filter paper. Recent investigations, however, have demonstrated the importance of a division of these two classes into three groups, the settling, colloidal and crystalloidal solids. In many respects the suspended colloidal solids behave in a manner similar to the colloidal solids in solution, and while a portion of them may be removed from sewage by prolonged sedimentation this fact is not generally sufficient to make it necessary to consider them independently of the colloids in solution.

The organic matter usually constitutes that portion of the sewage

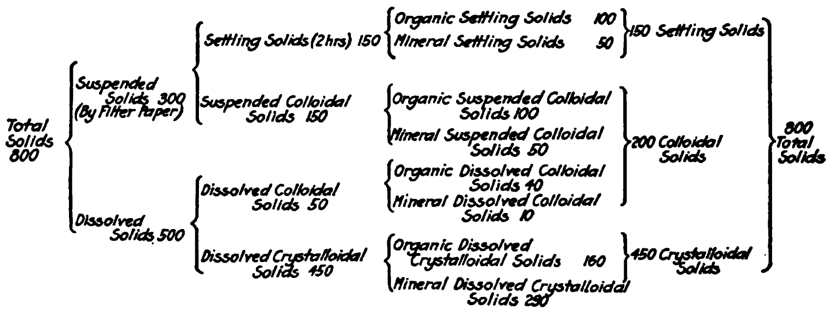


FIG. 16.—Physical condition of principal constituents of sewage of medium strength. (Numbers are parts per million.)

which causes trouble, and if it can be removed or converted into mineral matter, the problem of disposal may be said to have been solved.

Where sewage is treated artificially to deprive it of its objectionable characteristics, one or both of two fundamentally different processes are generally employed: first, the actual removal from the sewage of certain constituent parts, as, for example, when it is passed through screens and coarse matter is temporarily retained on the screens and later removed from them and buried or burned; second, the conversion of putrescible organic matter into stable substances.

The treatment of sewage is frequently accomplished by a series of steps, the degree of purification attained increasing with each successive step. Where the discharge of raw sewage will be only slightly objectionable, a single process, such as screening, may suffice. If conditions require the removal of more matter, sedimentation may be added, and if the dissolved putrescible organic matter must be removed from the sewage before its discharge, some type of filtration by which it may be oxi-

dized may follow the sedimentation. If conditions are such as to require the practical absence of pathogenic organisms from an effluent, disinfection may be the final process. In such a case, the treatment would involve four successive steps, screening, sedimentation, oxidation by filtration and disinfection.

SEWAGE DISPOSAL BY DILUTION

Sewage disposal by dilution is the discharge of sewage into natural waters where, if successful, it will be dispersed through the waters in such a manner as to be carried away so rapidly, or be so changed in composition and character, that it will not prove offensive or a menace to health. In America it is common practice to dispose of sewage by dilution in rivers, lakes and tidal waters. The active forces of purification are physical, chemical, bacteriological and biological. The principal physical forces are sedimentation and dilution; the chemical, biological, and bacteriological processes are complex, and authorities are not agreed as to the exact nature of the actions or the relative importance of the various agencies. The transformation of putrescible organic matter into more stable organic matter and finally into inorganic and inert compounds, nitrates, carbon dioxide and water, is a well-known and fundamental change. The problem of sewage disposal by dilution is to bring about the change without causing nuisance.

Floating and Suspended Matter.—In considering disposal by dilution it is particularly important to pay attention to the suspended solids, for while similar in composition to the dissolved substances, they are subject to somewhat different laws and actions. It is also of fundamental importance to remember that organic matter constitutes food for living organisms, as pointed out in the introduction to Chapter IV. Much of the suspended matter is not suitable food, being inorganic. There are probably rarely sufficient birds, fish and other food consumers present to remove all suspended food matter immediately, and a portion is carried along by the current or is deposited about the sewer outlet.

Velocity of Current.—The velocity of the currents into which sewage is discharged may have an important bearing upon the results of such discharge. If sewage be discharged into a swiftly moving, relatively small stream, the suspended matter will be carried away from the community so rapidly that it will not have an opportunity to form deposits or to putrefy and cause offensive conditions within the community where it originates. When the stream reaches a lower riparian town its slope may be flattened, with consequent reduction of velocity, thus allowing it to deposit a part of the suspended matters it has been carrying and affording time for the organic matter to decompose so as to produce offensive conditions. Such deposits may tend to create a nuisance as well as to cause objectionable shoaling.

Sedimentation is also important because by it a very substantial proportion of the organic matter of sewage may be removed from the diluting waters and retained at the bottom of deep pools in rivers, lakes and tidal waters. The decomposition of such sludge deposits is relatively slow. Thus, in some rivers, sedimentation will remove the settling solids from the waters, throwing upon the rivers principally the burden of the dissolved and colloidal organic matter. In time of freshet, when there is ample volume of water and swift currents, such deposits may be scoured out and the rivers relieved of the burden of changing this organic matter to mineral or more stable organic substances. Under such conditions sedimentation in the river performs a function similar to that of settling tanks, through which the sewage may be passed before its discharge.

Decomposition of Organic Matter.—If the proportion of sewage to diluting water is large, the bacteria may thrive to such an extent that their demand for oxygen will exhaust the available supply and anaerobic processes will set in, followed by the offensive conditions of putrefaction. In any event, such processes are likely to go on in the sludge banks formed by the precipitation of organic sewage matters. If, on the other hand, there is enough oxygen to meet the demands of the bacteria, the aerobic organisms will predominate and the organic matter will be oxidized, as already explained in Chapter III.

By bacterial oxidation the organic matter is converted into simpler compounds, some of which are suitable for plant food, and the oxidation of sewage under conditions favorable to the growth of plants is certain to be followed by such growth, as described in Chapter IV. Just what the functions of the plankton are under all conditions is not clear, but it is certain that the oxygen exhaled by them is an important factor in maintaining the supply of dissolved oxygen in water under certain conditions. It is not uncommon to see the beds of small streams, which receive relatively large quantities of sewage, covered with green algæ from which vast numbers of little bubbles of oxygen gas are given off, and such waters are not infrequently so charged with this oxygen that they are supersaturated. Similar processes occur in ponds and lakes, although in such cases the supersaturation is usually confined to the upper portion of the water. This is because the plankton thrives at the surface where the light is strong and supersaturates that portion of the water, while bacteria thrive in that below, making large demands upon the dissolved oxygen, which is still further depleted by contact with the products of anaerobic mud decomposition at the bottom. In muddy rivers the growth of plankton is quite different from that in relatively clear waters. This is well illustrated by the Ohio River, in which at low water, when the sediment has settled and the water is

fairly clear, the plankton grows rapidly, but when the water is muddy its growth is greatly retarded.

Temperature.—The effect of temperature upon sewage disposal by dilution is more important than is sometimes recognized. A river may receive in winter, without creating objectionable conditions, a quantity of sewage which, in summer, would cause it to be a rank nuisance. Bacterial action at low temperatures is relatively slow, and sewage may be carried by the stream receiving it to tide water, or to a point where ample dilution is afforded, before there is enough bacterial development to cause objectionable conditions. This has an important economic bearing, for, in summer, sewage treatment may be carried to a degree insuring satisfactory conditions at an expense which, if continued through the year, would be prohibitive; advantage may be taken of the winter conditions by providing a less complete treatment, care being taken to avoid sludge deposits which may prove objectionable during the succeeding warm season.

Importance of Dissolved Oxygen in Diluting Waters.—An ample supply of dissolved oxygen must be constantly present in natural waters receiving sewage, if putrefactive conditions are to be prevented. The two sources from which the supply can be renewed are the atmosphere and the plankton. The latter is most effective in the northeastern part of this country during August and September.

Absorption of oxygen from the atmosphere is usually the chief source of dissolved oxygen in water. It is this ability of water to absorb oxygen from the air rapidly which has maintained the purity of most ponds, lakes and oceans in spite of pollution and the very large amount of organic matter washed into them from the surface of the earth. Moreover, there is no evidence of deterioration of the quality of these waters except in isolated cases, where the digesting capacity of the water has been exceeded. The rapidity with which a water whose dissolved oxygen has been reduced by sewage oxidation will absorb air from the atmosphere varies greatly under different conditions, as explained in Chapter VII.

Sewage Treatment Necessary because of Limitations to Disposal by Dilution.—To prevent putrefactive conditions in water it is often necessary to resort to the treatment of the sewage prior to its discharge. If the floating matter alone is objectionable, it is necessary only to remove that. If deposits are the source of complaint, the removal of the settling solids may be enough. Where dilution fails because of lack of oxygen, the treatment of the sewage may be carried far enough to reduce its oxygen demands sufficiently to enable the natural water to provide an adequate supply of oxygen. It is essential, before determining the exact type of treatment to be adopted, to ascertain what the requirements of the situation are and to what extent it is necessary to remove the objectionable constituents from the sewage.

GRIT CHAMBERS

Most sewage contains some heavy mineral matter like sand, gravel, bits of coal and cinders. The first flushings from streets in times of storm carry large quantities of such material into combined sewers, and it finds its way into separate sewers through perforated manhole covers, carriage washstands, cellar drains and occasionally it is actually thrown into water-closets. Such matter may cause deposits in siphons and settling basins and wear of pumps. While it may form deposits in rivers and lakes, it has not been sufficient in quantity ordinarily to require its removal where no other form of treatment has been necessary. The removal of this material, commonly called grit because of its sandy nature, has proved necessary for the protection of machinery, siphons and treatment works rather than because of the difficulty in disposing of it by dilution under most American conditions.

Its removal may be accomplished by sedimentation in small basins, called grit chambers. The chamber is simply an enlargement in the sewer, so proportioned as to retard the velocity of the stream of sewage. Grit can reach the outlet only when the velocity of flow of the sewage is sufficient to carry it along, in general, 2½ to 3 ft. per second. If the grit is to be removed from the sewage by sedimentation, its flow must fall below this critical velocity.

Grit settles from flowing sewage according to its size and specific gravity. A velocity which will just permit pebbles ¼ in. in diameter to settle will carry along sand. Another element entering into the problem of removing grit is the depth of flow through which it must settle; upon this will depend the length of the grit chamber, for a definite period of time is required for a particle of given size and weight to fall a specified distance through a flowing stream. The grit chamber, therefore, must be long enough to provide time for the particle to reach a depth below which it will be prevented from passing out.

It is important at the outset to decide on the character of material which shall be removed by the grit chambers. Under some conditions, it may save expense in operation to remove from the sewage not only the coarse grit, like pebbles and masons' sand, but also fine sand and some of the heavy organic matter. If permitted to pass on with the sewage, these would cause large deposits in settling tanks, which would not flow out with the sludge and would, therefore, entail large expense for removal. In other cases, the removal of pebbles, sand and cinders, as nearly free from organic matter as possible, may prove preferable. The organic matter likely to be precipitated in grit chambers is extremely offensive if the velocity of flow be reduced below 1 ft. per second, and wherever practicable it is more satisfactory to handle it with the sludge from settling tanks than to attempt its removal by grit cham-

bers. Unfortunately, it is difficult to design and build grit chambers which will remove an adequate proportion of the grit without at the same time retaining more or less highly putrescible organic matter. Coffee and tea grounds, raisin and fruit seeds, rags, paper, bits of meat, vegetable matter and feces settle readily at fairly high velocities and may be expected in the sediment retained in grit chambers, unless the velocity is maintained so high as to permit the sedimentation of only very coarse sand and gravel.

To provide a uniform separation of grit from other settling solids, a uniform velocity must be maintained. This is difficult on account of the constant variation in the quantity of sewage and the large capacity of the chambers in proportion to the flow during the early years of their service. This difficulty may be overcome, in part, by building grit chambers in such units that their number may be increased from time to time as the flow increases. The difficulties of operation will be reduced by retaining in the grit chamber only such matters as will settle at relatively high velocities, not lower than 1 ft. per second, if it is practicable to handle the remaining heavy material in other portions of the treatment plant. Consideration may well be given in such cases to determining the real value of removing the small quantity of grit which can settle under such conditions, for it may prove that the grit chamber is accomplishing no useful purpose, particularly with separate sewerage systems. Frühling estimates that in a sewage containing about 700 parts per 1,000,000 of suspended matter, about 115 parts per 1,000,000 can be removed by grit chambers. This is very strong sewage, from an American viewpoint, and from 10 to 40 parts per 1,000,000 are likely to conform better to the conditions in the United States. In considering the percentage removal of suspended matter by grit chambers, it should not be forgotten that some of the material thus removed is so large that it is not collected in samples and is not included in the results of analyses.

SCREENS

Municipal sewage contains cloth, paper, lumps of meat, bits of vegetables and pieces of wood from the size of a match to that of a railway tie. Some of this refuse will interfere with pumping and other machinery, and much of it may prove unsightly where sewage disposal is by dilution. Floating refuse also gives to open settling tanks a disagreeable appearance, and where sewage has not undergone disintegration the coarser suspended solids, such as paper, hair, cloths, waste and other fibrous matter, may assist in forming a tough, troublesome scum which floats upon the surface of the sedimentation tanks. Hence it may be desirable to remove some of this matter. Two classes of screens are used for this purpose: coarse racks capable of removing

material likely to obstruct or injure machinery, to render diluting waters unsightly, and to prevent deposits of very coarse matter upon the beds of rivers, lakes and tidal waters; and fine screens, which may remove a much larger proportion of these substances. Coarse screens are usually built of parallel bars, seldom less than ½ in. apart. Fine screens are commonly built of perforated metal or wire cloth, the perforations rarely exceeding ¼ in. in size. Sewage should not pass screens at very high velocities unless the object is merely to retain very large materials which may prove injurious to machinery, for much of the organic matter will be macerated and forced through the screens by impact and the attrition of the sewage.

It is sometimes maintained that fine screens perform similar work to that accomplished by settling basins. While most of the matter removed from sewage by fine screens would be retained in sedimentation tanks, it is not a fact that all of the material capable of such retention can be removed by fine screens. Dr. George A. Soper states his conception of the facts as follows:

"The results of operation, under the best circumstances, are quite different with screens and settling basins, and although some of the tests which have been reported seem to indicate that they may both do about the same work in the removal of organic matter in sewage, a moment's thought will show that they produce different effects on the sewage. Screens remove particles according to size, settling basins according to weight. Neither approaches the ideal, which is removal according to composition. Screens, however, have some advantage, inasmuch as a large part of the heavy solid matter which goes to the credit of settling basins is not putrescible. On the other hand, much of the material which floats will not decompose rapidly enough to make trouble if left in the sewage, so that the difference between screens and settling basins on this account is not great. Screens have the disadvantage of breaking up some of the very material which it is most desirable to remove, so that it has been claimed that there was more organic matter in solution in the effluent than in the applied sewage in some plants. There is no such action in settling basins, but the latter have a compensating disadvantage in the fact that they keep the sewage on hand several hours longer, thus favoring decomposition." (*Proc. Am. Soc. C. E.*, vol. xli, page 656.)

There appears to be a great difference of opinion upon the proportion of floating and suspended matters removed from the sewage by screens. The available data are quite unsatisfactory because procured by a diversity of methods and under various conditions. Furthermore, it is not possible when taking samples of unscreened sewage for analysis to make them fairly representative of the coarser and heavier matter in the sewage. Accordingly, when the weight of screenings is computed into parts per million of suspended matter in the original sewage, it is quite likely that little of it was included in the suspended matter reported from analysis of the unscreened sewage.

From such data as are available, it appears that coarse screens will remove from less than 5 to perhaps 10 parts per 1,000,000 of coarse material and that fine screens may be expected to remove from 20 to 70 parts per 1,000,000, where the sewage is strong and fresh and where conditions are otherwise favorable for screening.

The disintegration of suspended matter during its passage through sewers and pumping machinery has an important bearing upon the efficiency to be expected from a screening plant. In this way much of the matter is converted into finely divided suspended matter, and some even into a colloidal condition, which enables it to pass the screen. It is important, therefore, when fixing the size of screen openings to give careful consideration to the nature of the sewage which must be screened, and under certain conditions it may be wise to provide for screening the sewage before it has had time and opportunity to become disintegrated and colloidal.

While screens will remove some of the suspended and floating matter, there will always remain large quantities of such substances to be discharged into diluting waters or to be removed by subsequent tank treatment. Screening will not reduce the dirty, greasy appearance of sewage.

PLAIN SEDIMENTATION

While grit chambers and screens will remove the heavy, sandy matter and the larger floating and suspended objects in sewage, there are many cases where it is advisable to remove still more suspended solids in which there is much organic matter that must be oxidized in the general process of disposal. If discharged into water it is liable to form sludge deposits. It tends to clog fine-grained filters and to impose an unnecessary burden upon coarse-grained filters. Disinfecting chemicals cannot readily gain access to solid particles of organic matter, and, therefore, the process of disinfection may be rendered relatively inefficient if the sewage contains large quantities of suspended solids. It is desirable, therefore, where disinfection is to be employed to remove first from the sewage the suspended matter which can be readily precipitated in settling tanks.

Sedimentation is the process by which sewage is allowed to stand quiescent in or to flow very slowly through tanks in which the suspended solids capable of settling under the existing conditions gradually subside to the bottom where they accumulate as a thick mud, commonly called sludge. The term "plain sedimentation" is used to distinguish the simplest form of this process from modifications of it intended to encourage bacterial action in the sewage or the sludge, or to remove colloidal matters which will not settle unaided in from 2 to 8 hours.

Work to be Performed by Sedimentation.—In the hypothetical analysis, it has been assumed that the suspended solids comprise 300 parts per 1,000,000 of which 150 parts are capable of settling in 2 hours. Still greater portions are capable of settling in longer periods of time. By passing such sewage through grit chambers and screens, it appears that from 15 to 110 parts per 1,000,000 suspended matter may be removed, thus leaving from 40 to 135 parts suspended matter capable of settling in suitable tanks. But it is probable that much grit and coarse suspended matter in the original sewage would not be reported as suspended matter in an analysis, and it is safe to assume, therefore, that a large portion of it is not included in the 300 parts per 1,000,000 assumed in the hypothetical analysis. Therefore, a large portion of the 150 parts will pass through the grit chambers and screens.

In studying the efficiency of sedimentation, it is important to consider whether the object of the investigation is to determine the percentage efficiency of the tanks in removing settling solids or their absolute efficiency in preparing sewage for dilution or further treatment. In the latter case it is important to consider the actual quantity of suspended solids remaining in the tank effluent.

Detention Period.—When sewage is allowed to stand quiescent, the heaviest suspended matter is first precipitated. This is followed by portions which are successively lighter than those previously precipitated. After prolonged standing, a part of the suspended colloidal matter may be thrown down gradually through coagulation, due to physical contact or chemical and bacteriological changes going on in the sewage, and in due course a portion also of the dissolved colloidal matter may be similarly precipitated. It is generally impracticable to provide storage basins of sufficient size to retain sewage long enough to provide for the precipitation of colloidal matter. Even were they financially practicable, the decomposition of sewage and sludge would generally render such a process inadvisable.

The quantity of suspended matter precipitated in various periods of time should be ascertained if possible and that period of sedimentation chosen which will cause the removal of the largest quantity required by the conditions. Recent studies indicate that such a period may be very short, perhaps 30 minutes in some cases, and that it is unlikely to exceed 4 to 6 hours. In the Emscher District many sedimentation plants have been built to provide a detention period[1] of 2 hours,

[1] The detention period is the length of time assumed to be required for the influent to displace a tankful of sewage. That is, if a tank holds 100,000 gal. and the rate of sewage flow is 2,400,000 gal. in a day, the detention period is 1 hour. This is called the "Durchflusszeit" or "flowing-through time" by German engineers. There seems to be no well-established practice about including or excluding the space occupied by the sludge, which in the older types of tanks is a variable and substantial factor. The sludge compartment is naturally excluded in the case of the Imhoff tank.

upon the theory that all suspended matter which need be removed from sewage before its discharge into the natural water-courses of the district will settle in that time. In certain experiments at Cologne, Steuernagel found that 72.3 and 58.9 per cent. of the settling solids were precipitated in tanks through which the sewage was allowed to flow in a horizontal direction at rates of 47 and 472 ft. per hour, respectively. These rates corresponded to detention periods of 187.5 and 18.75 minutes.

The first object of the designer of such basins should be to provide conditions enabling the particles of suspended matter to settle quickly. Formerly this was sometimes done by filling a tank with sewage, allowing it to stand quiescent for a given period of time, and then drawing off the supernatant liquor. Dr. W. P. Dunbar states the disadvantages of this method as follows:

"Theoretically, intermittent action in which the sewage is allowed to come to rest is more efficacious than continuous action in which the sewage is allowed to flow continuously through the tanks. Continuous action has many practical advantages over the intermittent method of working. At each emptying and filling of the tank there is a danger of stirring up the sludge which should, therefore, be removed each time the tank is emptied. . . . Intermittent action also causes a loss in available head of the sewage equal to the height in the tank, and the time of filling and emptying are not utilized in the purification process." ("Sewage Treatment," page 65.)

He cites the experience of Santo Crimp in London, who obtained much more sludge by continuous than intermittent operation. This is not in accord with the experience of the authors, who have found that the volume of sludge is generally increased by frequent removal, almost necessary when operation is by the fill-and-draw method, and that sludge allowed to remain in the tanks for a longer period of time, as is usual in the continuous-flow method, is usually much more compact and contains a much higher percentage of solids than that removed more frequently.

It is the almost universal practice to allow sewage to flow continuously through sedimentation tanks at velocities below 475 ft. per hour, the sedimentation efficiency being substantially as good as by the intermittent system, and the operation of the tanks being much more convenient and economical.

Size and Shape of Settling Tanks.—Some settling tanks are cylindrical, sewage entering somewhat above the bottom and overflowing at the top, Fig. 17. Sludge is drawn from the conical bottom without requiring the emptying of the tanks and may be delivered by gravity at an elevation considerably above the bottom of the tank. In such tanks, the upward flow must be less rapid than the downward velocity of most of the settling particles, some of which tend to collect in a layer or mass at some elevation between the influent orifice and the overflow weirs. Par-

ticles moving upward may be caught in this layer, which acts as a sort of filter and is said to aid the sedimentation in such tanks.

Other tanks are shallow, rectangular structures through which the sewage flows in a horizontal direction at velocities sufficiently low to permit the suspended particles to settle. The theory of sedimentation in horizontal flow basins is complex and is discussed at some length in Chapter X. The practical sedimentation of sewage is complicated by

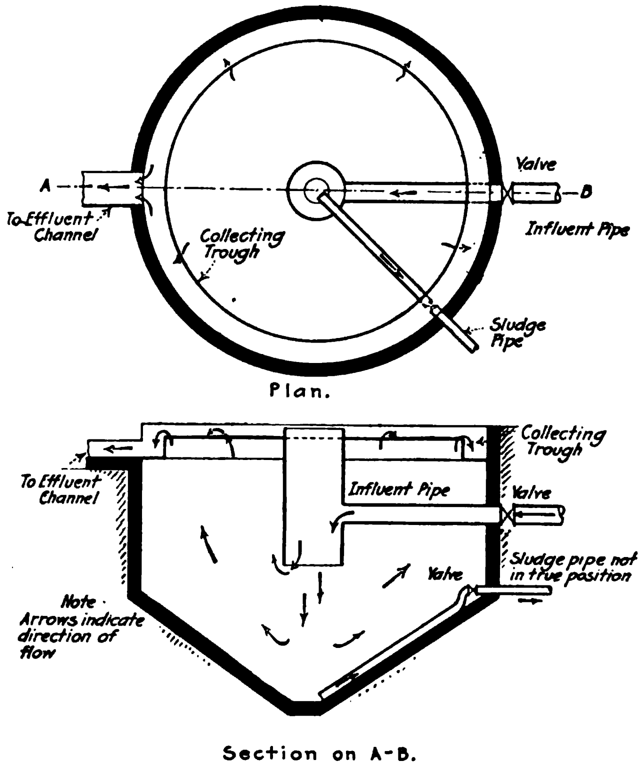


FIG. 17.—Secondary settling tank, Fitchburg, Mass.

the great diversity in the size, weight and specific gravity of suspended particles; by gases contained in and surrounding particles and rising from the sludge below; by wind action where tanks are open; by fluctuations in volume and temperature, and by variations in the composition and strength of the sewage.

The results of studies made at the Columbus Experiment Station were interpreted by George A. Johnson to indicate that the proportion of suspended matter removed from the sewage, under like conditions, is in a

general way inversely proportional to the quantity of suspended organic matter present. (Report on Sewage Purification, page 105.) The opposite is indicated by the results reported by Wisner and Pearse from experiments performed at the Thirty-ninth Street Experiment Station at Chicago and at the Chicago Stock Yards. (Report on Industrial Wastes, page 170.) Studies by the authors of results obtained with the operation of sewage treatment plants indicate clearly that the stronger the sewage, as measured by solids in suspension, the greater the percentage removal of suspended matter in the sedimentation basins.

It is customary to build tanks from 6 to 10 ft. in depth and with a ratio of width to length of from 1:2 to 1:6. Many tanks have bottoms laid horizontal in the direction of the flow. In some cases they are deeper at the outlet end, while Steuernagel suggested, from his experiments at Cologne, that they should be deeper at the inlet end.

The heavier and greater part of the settling solids subside quickly near the inlet end if velocities are not excessively high. It is, therefore, convenient to provide a somewhat greater depth at this end, so that the sludge may be retained as close as possible to the sludge outlet sluices, which should be placed at the inlet end of the tank. This will result in saving considerable labor where tanks are cleaned by scraping and squeegeeing the sludge, as is common in the older plants. The gates for drawing off the supernatant liquor, however, may well be placed at the outlet end of the tanks, where usually the sludge is not as deep as at the inlet end and it is frequently possible to draw the water lower than at the other end.

Scheme of Flow through Tanks.—Recognizing the fact that fine and light particles of suspended matter require a longer time in which to settle than the heavier particles, many of the older plants were designed to provide a long route through which the sewage was obliged to flow. This was frequently accomplished by arranging tanks "in series," like the original chemical precipitation tanks at Worcester, Fig. 18, built in 1889 after designs by the late Charles A. Allen. Here the sewage was admitted to the first tank at *a*, allowed to flow through the tank and out at *b*, then in the same way through each of the other tanks, unless one or more were cut out for cleaning. Relatively high velocities caused considerable agitation at the inlet and outlet weirs.

In 1893 additional tanks were built after plans of Frederick A. McClure, City Engineer, the method of operating which (in conjunction with the old tanks), is also shown in Fig. 18. The old tanks were used for roughing basins to collect the heaviest part of the sludge. The sewage passed through them in series but was admitted to the new tanks "in parallel," thus providing for a minimum velocity of flow. The latter method proved to be much more satisfactory, especially because it facilitated the handling of the sludge, a large portion of which settled in

the roughing tanks. These were cleaned at frequent intervals, at times as often as once in 2 or 3 days. The secondary or finishing tanks received relatively little sludge and did not require cleaning more often than once in 2 or 3 weeks to prevent active putrefaction in the sludge

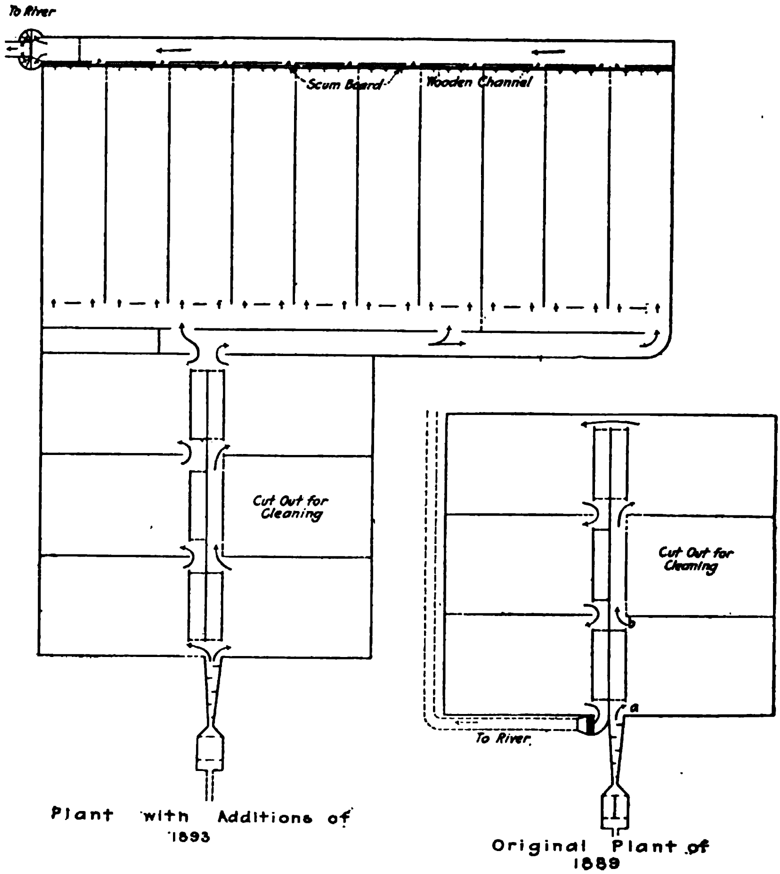


FIG. 18.—Sedimentation tanks at Worcester, Mass.

during warm weather. In winter they were sometimes used for 5 or 6 weeks without being cleaned.

More recently sedimentation plants have been built in such a way as to provide for each portion of the sewage to pass through but a single tank. This type of construction is illustrated by Fig. 83, plain sedimentation tanks at Marlboro, Mass., and Fig. 98, Imhoff tanks at Fitchburg, Mass. It is sometimes found desirable to provide transverse

walls or sludge dams, as at Marlboro, to retain the sludge near the gates through which it is removed.

Number of Basins.—It is desirable even in small plants to build duplicate settling basins, so that there may be no complete interruption in the treatment of the sewage. In larger plants the number of units to be provided will depend upon structural considerations and in the case of shallow tanks, like those at Worcester, upon the facilities for removing sludge. It is also desirable to provide a number of relatively narrow tanks, rather than one or two very wide tanks, to minimize the effect of wind and to aid in establishing rates of flow throughout the width of the basins which are reasonably uniform.

SEPTIC TANK

About 1895 Donald Cameron constructed a small, tightly covered experimental tank, mentioned on page 14, through which he allowed sewage to pass slowly, the inlet and outlet pipes being submerged. His intention was to provide an opportunity for the liquefaction of the suspended organic matter of sewage by anaerobic organisms working in the dark.

The sludge was allowed to remain in the tank, which he called a "septic tank." The process of sedimentation coupled with conditions which allow the sewage to undergo anaerobic decomposition in contact with decomposing sludge has come to be known as the "septic process."

The primary function of this tank is that of sedimentation, the detention period varying in different places, being in general from 8 to 24 hours. This period is so long that some of the colloidal solids are precipitated, either through contact with the sludge and surfaces of the walls, or through biological changes.

Anaerobic Action.—The septic process, aside from the physical sedimentation of the solids, depends upon anaerobic bacterial action, described on page 101. It was formerly considered essential to the most vigorous development of this action for it to be carried out in the absence of atmospheric oxygen and in the dark, thus providing ideal anaerobic conditions. Experience has shown that this was not so, but there are other reasons, mentioned in Chapter XI, which sometimes lead to roofing septic tanks. The bacterial action is similar to, and a continuation of, that going on in sewers. When the tank is first filled, the bacteria in the sewage increase rapidly in numbers, any dissolved oxygen or nitrates which may have been present are quickly exhausted, and anaerobic decomposition is set up. This is accompanied by the decomposition of the complex organic matter, with the formation of ammonia, carbon dioxide and other relatively simple compounds. The suspended matter which accumulates at the bottom of the tank as sludge is also attacked by organisms, and the solids are disintegrated,

coarse matter being transformed into finely divided particles. This action converts some solid organic matter into gases, as carbon dioxide, methane, nitrogen and hydrogen, or into soluble substances which pass out of the tank dissolved in the effluent.

One of the most obvious features of the septic tank is the production of these gases, which are generated in large quantities in the sludge. There they are held mechanically until they make portions of it so buoyant that they suddenly break away from the remainder and are carried through the supernatant liquid to the surface. This action at times becomes so violent as to give the tank the appearance of ebullition, and a large proportion of the accumulated sludge is temporarily carried to the surface, where much of the gas may be liberated and the solid particles, thus being deprived of their support, may again be precipitated.

At temperatures of 15° to 20°C., such conditions may become established in from 3 to 6 weeks. If, however, a new tank be seeded with a small quantity of sludge from an active septic tank, the "ripening period" required for developing activity can be materially reduced.

Oxidation of Septic Effluent.—It has been held by some that the anaerobic breaking down of complex organic substances makes sewage more susceptible to bacterial oxidation in filters. Experience has failed to demonstrate this clearly, and the consensus of opinion at present appears to be that municipal sewage after septic action is not more easily oxidized than the fresh sewage. Furthermore, there are strong indications that under certain conditions over-septicization occurs and substances are formed which are more or less inimical to the bacterial life upon which oxidation depends. It is also a fact that the septic effluent generally has a much greater avidity for oxygen than fresh or stale sewage.

These difficulties can be overcome to some extent by a thorough aeration of the septic effluent. In this way its avidity for oxygen may be partially satisfied, thus relieving the oxidation process of a part of its burden, and gaseous compounds unfavorable to bacterial life may be liberated. The effluent from septic tanks is more likely to be offensive than fresh sewage, and such aeration is frequently attended by the production of offensive odors.

Sludge Digestion and Consolidation.—It is obvious from the evolution of gas in the septic tank that the organic matter in the sludge is undergoing such changes that a reduction in its quantity must follow. The early belief of some enthusiasts was that perhaps most of the sludge could be liquefied by this process. However, measurements indicate that the reduction in weight of solid matter varies in different places from 10 to 40 per cent., averaging perhaps 30 per cent., and there always remains a substantial quantity of sludge.

The increase in density of the sludge due to its disintegration and solidification, resulting from a prolonged stay in the tank, causes a great reduction in volume. The sludge removed from the septic tank probably does not exceed 20 to 25 per cent. of the volume removed from plain sedimentation tanks so operated as to prevent septic action.

There is much difference in the character of sludge from different septic tanks, due to variations in quality of sewage, temperature, construction of tanks, methods of operation, and other local conditions. Some sludges are said to have been practically free from odor, but there are many others which have been decidedly offensive.

Scum.—A noticeable feature of some septic tanks is the formation of scum. This consists of the coarser suspended matter which floats either on account of its light specific gravity or because it is buoyed by large quantities of gas. The quantity of scum formed depends largely upon the character of suspended matter. If the sewage is fresh and the suspended matter not much disintegrated, large quantities of scum are probable. The suspended matter brought to the surface of the sewage forms such a compact mass that the entrained gases can be liberated but slowly. Meantime the formation of more gas in the remaining sludge carries more suspended matter to the surface, increasing the thickness of the scum perhaps to 2 ft., and it often projects 2 to 6 in. above the surface of the sewage. Under such conditions, especially in open tanks, the surface of the scum is likely to become dry and leathery, thus forming a fairly tight roof, sometimes cracked by the gas pressure below it.

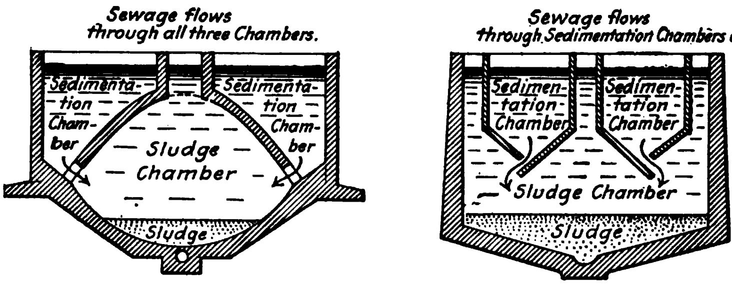
Results of Septic Treatment.—While the settling solids and a portion of the colloids may settle in a septic tank at first, the violent ebullition often carries large quantities of finely divided solids of the sludge into suspension, and thus causes them to be carried out with the effluent. This diminishes the efficiency of this type of tank as a means of sedimentation. The effluent frequently possesses an offensive odor, has a greater avidity for dissolved oxygen than fresh or stale sewage, and may contain substances inimical to oxidizing bacteria.

TRAVIS HYDROLYTIC TANK

In the tank originated by Dr. William Owen Travis, which was briefly explained on page 21 and is described in detail in Chapter XI, it was proposed to avoid the contact of sludge with the greater portion of the sewage and thus prevent it from absorbing those substances which produce offensive odors and tend to make it more difficult to oxidize. This was done by allowing most of the sedimentation to take place in two chambers, Fig. 19, from which the sludge passed through openings into a third chamber, through which but little sewage flowed, where the anaerobic decomposition of the solids took place.

IMHOFF TANK

Another step in digesting sludge apart from the sewage in the sedimentation tank has been taken by Dr. Karl Imhoff, who has built a large number of two-story tanks, Fig. 19, in the Emscher River District of Germany. This type differs from the Travis tank in allowing no sewage to flow through the sludge digestion chamber, as explained on page 23 and described in detail in Chapter XI. Furthermore, Imhoff recognized the importance of passing sewage through sedimentation tanks as quickly as possible, to avoid exhausting the supply of dissolved oxygen or nitrates, thus preventing the formation of offensive odors and conveying the sewage as rapidly as practicable to the river or the treatment plant. These two features of treatment have proved of marked practical importance. This process differs, therefore, from the septic tank process in the effort made to keep the sewage as fresh



Travis Tank — Imhoff Tank

FIG. 19.—Types of two-story tanks.

as possible and not to exhaust its dissolved oxygen, instead of calling putrefactive action into play.

The only function of the upper compartment is that of removing the settling solids in the sewage. These solids pass into the lower or sludge digestion chamber, where the sludge remains for several months to undergo biological decomposition. Large quantities of gas are evolved and scum is found upon the surface of the sewage in the gas vents. No gas can enter the sedimentation chamber on account of the form of the slot at its bottom. After the sludge has remained in the sludge chamber for several months and has been digested by the bacteria, the bottom portion when drawn off is found to be filled with gas, to contain from 75 to 85 per cent. water, and in some cases to have a tarry or creosote odor not likely to cause offense under usual conditions. In some cases the digestion of the sludge in such a manner as to assure absence of offensive odors is stated not to have been achieved.

Processes in the Sludge Chamber.—The processes in the sludge chamber are similar to those of the septic tank. The solids accumulate in this chamber continuously rather than in large volumes intermittently, as in separate sludge digestion tanks, a difference which, it has been suggested, may have an important influence upon the digestion process.

As in all sedimentation and septic tanks, the sludge retained in the sludge compartment for a long period tends to become more compact; its water content is decreased and also its volume. This is due to physical consolidation and to biolytic disintegration of the coarser suspended matters, which, in their original condition, cause many relatively large voids in the sludge, which are filled with water.

When sludge digestion is proceeding satisfactorily, the sludge withdrawn from the tank will contain large quantities of gas held mechanically in it. This is an important feature of the process, for when relieved from the pressure of the supernatant sludge and liquid in the tank and spread out upon the sludge-drying beds, the solid matter will be buoyed up by the entrained gas, thus permitting the relatively clear water below to pass quickly into and through the beds. As the water disappears, the sludge gradually settles down on the surface of the bed, and the gas is liberated and replaced by air, leaving the mass quite porous and in a condition favoring rapid drying.

In his book upon "The Operation of Sewage Disposal Plants," F. E. Daniels describes satisfactory Imhoff sludge in the following terms:

"Good sludge should be dark in color, more or less granular and not sticky or pasty, should not have offensive odor, should be somewhat frothy and a good deal like black garden soil mixed with water. If a pailful be poured out and the pail put in an inclined position, what was in contact with the surfaces of the pail will separate and clear streaks will appear upon the metallic surfaces. Bad sludge is only partly decomposed, is usually lighter in color, has offensive odor, does not dry rapidly, and is somewhat sticky or pasty. This should not be withdrawn but should be kept in the tank to ripen" (page 35).

Very little is known of the causes of the bio-chemical changes going on in the sludge digestion chamber, beyond the general features of anaerobic action outlined in Chapter III. In some cases fermentation has been very active, attended by the production of large quantities of gas. This has occasionally caused the formation of much floating sludge buoyed up by the gas, even to the extent of causing an overflow of gas-filled scum from the gas vents. In many cases the digested sludge has been practically free from offensive odors, but there have been a number of cases where it has been sour and extremely offensive. These different results of digestion are undoubtedly due to certain species of organisms gaining the ascendency in the digestion chamber in some cases,

while others have predominated in cases where different conditions have been observed. Apparently more favorable conditions exist where there is a supply of alkali to combine with the acids as they are formed. This requirement is met sometimes by the natural and ammoniacal alkalinity of the sludge, and in other cases by the addition of lime. Clean water introduced into the sludge compartment often has been beneficial, but its advantageous effect may have been due to alkalies contained in it. Daniels gives his theory of the cause of foaming as follows:

"The filling (when first starting the tank) completely with such a strong sewage put so much matter in the sludge compartment that, when violent ebullition occurred, the liquids were so viscous that the gas bubbles would not break and a froth resulted—the whole mass 'working over' much like a barrel of cider. After the tank has 'worked' itself out, the supernatant liquid loses its viscosity and the gas bubbles break without causing any foam. Then as new matters come in they are not added fast enough to upset this equilibrium, and in this regard the tank takes care of itself. If, however, it should be drawn down very low and refilled with raw sewage, a repetition is likely to occur, as will be shown presently." ("Operation of Sewage Disposal Plants," page 38.)

At Atlanta, Ga., foaming is considered by Hommon to indicate the desirability of drawing off some of the sludge.

The known presence of sulphur in sewage sludge has given rise to the expectation that hydrogen sulphide would be formed in Imhoff sludge digestion chambers, as in many septic tanks. The absence of its odor in Imhoff sludge in the Emscher district has been attributed to iron salts in the sewage admitted to the tanks there. This seems reasonable, as iron has a strong affinity for sulphur, forming sulphide of iron, and may thus prevent the escape of hydrogen sulphide gas.

Results of Sedimentation.—The practical detention period appears generally to be not over 4 hours. This will permit the subsidence of a large proportion of those solids capable of settling in a practicable period, or about 45 to 65 per cent. of the total suspended matter. In the hypothetical sewage, Fig. 16, the settling solids are assumed to amount to 150 parts per 1,000,000, of which perhaps 130 parts may be assumed to be practically removable by plain sedimentation. This assumption is based on a tank efficiency of about 90 per cent. If the sewage becomes septic the ebullition of gases lifts the sludge, and the removal of suspended matter by sedimentation is counteracted to some extent in this way. The effluent will contain suspended colloidal matter, which together with the remaining settling solids will amount to about 170 parts per 1,000,000, will have a dirty turbid appearance, and, if the detention period is short and sludge is not allowed to become septic in contact with

the sewage, will have substantially the same odor as when it entered the tanks.

TANKS WITH COLLOIDERS

Quantity of Colloidal Matter.—In the efforts of the last quarter century to purify sewage, the accomplishment of different processes has usually been measured by the difference between the composition of the sewage and effluent, rather than by following the several constituents of the sewage through the processes to ascertain how each was affected. By thus dealing with aggregates, the colloidal matter was lost sight of until recently, although it constitutes a large and troublesome portion of the solids. In the hypothetical analysis on page 200, the colloidal substances constitute 200 parts per 1,000,000 and the effluent from plain sedimentation tanks may be assumed to contain settleable and colloidal solids as follows:

Settleable solids	15 parts per 1,000,000
Suspended colloids	150 parts per 1,000,000
Dissolved colloids	50 parts per 1,000,000
Total settleable and colloidal solids	215 parts per 1,000,000

These substances constitute 26.9 per cent. of the total solids of the sewage, of which about 19 per cent. or 150 parts per 1,000,000 is organic matter.

During the last 15 years, the colloids in sewage have been studied by several investigators, among them Dr. William Owen Travis, author of the Hampton doctrine, which is:

"The natural process of sewage clarification is, in the main, the result of physical operation, and bacteria play a subservient part in the purification." ("Elimination of Suspended Solids from Sewage," Jones and Travis, *Proc. Inst. C. E.*, vol. clxiv, 1906.)

He gave prominence to the solution of suspended matter during the passage of sewage through sewers and pumps and to the desolution of this matter by surface attraction. In this paper reference was made to a notable paper by Thomas Graham, the founder of colloid chemistry, in which the colloidal condition of matter is described as follows:

"A tendency to spontaneous change, which is observed occasionally in crystalloids, appears to be general in the other class (colloids). The fluid colloid becomes pectous (curdled) and insoluble by contact with certain other substances, without combining with these substances, and often under the influence of time alone. The pectizing substance appears to hasten merely an impending change. Even while fluid, a colloid may alter sensibly,

from colorless becoming opalescent; and while pectous, the degree of hydration may become reduced from internal change. The gradual progress of alteration in the colloid effected by the agency of time is an investigation yet to be entered upon." (*Phil. Trans. Royal Soc.*, 1861, vol. cli, page 183.)

The following quotation from Jones and Travis' paper describes the process to which they largely attribute desolution. Having described the sedimentation of settling solids, they state:

"The liquid portion (of the sewage) has now lost its thick appearance, though it is still mucilaginous, or opalescent, and of a yellow color. On looking carefully through the glass, no particulate matter appears to be present. After some time, however, very fine specks can be noticed just inside the glass, which gradually become plainer. The matter does not exactly form a film on the glass—though this is seen occasionally—but seems to collect together more or less in points; to use Graham's term, the fluid colloid has now become 'pectous.' These points gradually become larger and fall. The agglutinating or coagulating matter separates only at the circumference of the bottle, that is to say, where the liquid is in contact with the glass, and not in the main body of the liquid. The removal of the agglutinating matter from solution has an effect on the liquid, which gradually loses its mucilaginous appearance and passes through degrees of opalescence, until it finally becomes transparent. . . . The presentation of additional surfaces to the liquid, *e.g.*, one or more glass rods placed vertically in the vessel, results in deposition taking place upon these fresh surfaces, and in the falling of the deposited matters and their accumulation at the places where the rods rest on the bottom of the vessel.

"The factor which exerts the paramount influence in causing this separation of solid matter from solution, or pseudo-solution, is, therefore, surface attraction. The colloidal and other substances become particulate upon the surface toward which they are drawn, or with which they happen to come into contact. The value of presenting a larger extent of surface to the liquid cannot be exaggerated, for it not only causes increased deposition, but also shortens, in inverse ratio to the areas of the surfaces, the period during which the phenomena above described are observable."

Acting upon the theory thus described, Arthur E. Collins, with the help of Travis, constructed colloidal tanks at Norwich, England, Fig. 96, to carry the process of sedimentation farther than the removal of settling solids. Such tanks have not come into general use. There has been a feeling that while under some conditions a portion of the colloidal solids could be removed by such devices, the work accomplished was not likely to be sufficient to offset the expense of construction and some difficulties in operation. The time may come, however, when more attention will be given to this feature of sedimentation, as it is a subject which deserves investigation. Charles Brossmann has utilized the principle on a small scale in a number of plants, as described in Chapter XX.

CHEMICAL PRECIPITATION

Another method of removing colloidal matter is by chemical precipitation, which consists of mixing with the sewage one or more soluble chemicals and allowing the suspended matter to settle in sedimentation basins. Many substances have been used in this process. Some enter into chemical reactions with substances in the sewage by which the dissolved chemicals are thrown out of solution, others are inert and are intended to act merely to weight the precipitate and aid it in settling, while still others act chiefly as absorbents of gases held in solution in the sewage. The most common are calcium oxide or lime, CaO, sulphate of alumina, $Al_2(SO_4)_3$, ferrous sulphate, $FeSO_4$, and ferric sulphate, $Fe_2(SO_4)_3$, all of which enter into chemical reactions. Clay and soil have been added with chemicals to weight the precipitate and charcoal has been used for the purpose of deodorizing.

Theory of Action.—The theories upon which the several precipitants act upon the sewage differ in some respects, but, in the main, are fairly represented by the action of copperas and lime. The lime is added to the sewage to give to it the required alkalinity, after which the requisite quantity of copperas or ferrous sulphate is added. The chemical reaction is as follows:

$$FeSO_4 + Ca(OH)_2 = Fe(OH)_2 + CaSO_4$$

The soluble ferrous sulphate is thus converted into ferrous hydrate, which is relatively insoluble and forms a very bulky flocculent precipitate which appears to fill the sewage completely. When brought to comparative rest, however, the precipitate will be seen to separate quickly into masses or aggregates, which settle slowly to the bottom of the tank. The precipitate thus formed fairly surrounds and envelops the suspended colloids and mechanically carries them down with it. Such precipitates appear to affect the dissolved colloids and through absorption or other action carry a portion of them down also.

The action of different chemicals upon the several constituents of sewage has been little studied. Fowler states:

"Iron and aluminum salts have very little effect on anything but the fecal emulsion. Lime precipitates the soap, but a decoction of vegetables, such as constitutes a large part of kitchen wastes, was practically unaffected by any treatment." (*Jour. Soc. Chem. Ind.*, vol. xxx, page 1345.)

Results.—The degree of clarification depends largely upon the quantity of chemical precipitant used. If a sufficient quantity is added and the process is otherwise intelligently carried out, the resulting effluent will generally be clear, substantially free from suspended and colloidal matters, and when examined in a glass will have no pronounced color. In many cases, however, the quantities of chemicals

required for such a result would be prohibitively expensive; consequently effluents as ordinarily seen are not clear and do contain much colloidal matter. The percentage of suspended matter removed in actual practice in England ranges from 65 to 90; it has been a little over 80 in Worcester and Providence. Fowler draws the following very pertinent conclusion as to the practicability of chemical precipitation:

"The question of the use of chemicals is mainly an economical one, whether it is better to spend money in keeping suspended matters away from the filters, and so reduce the area of these and their upkeep, or whether the cost of chemicals and the disposal of the resulting sludge outweighs the filter cost. The question is one which must, like so many others, be decided in accordance with local circumstances, among which facilities for sludge disposal must always be a governing factor." (*Jour. Soc. Chem. Ind.*, vol. xxx, page 1345.)

One of the greatest difficulties of this method of treatment is the handling and disposal of the sludge, which is produced in great volume, often as much as 0.5 per cent. of the quantity of sewage treated. This difficulty and the attendant expense have been important factors in preventing the more general adoption of this method, and even in causing its abandonment in some places where already installed.

ACTIVATED SLUDGE

Attention was called in Chapter I to the possibility of clarifying sewage by forcing air through it and obtaining at the same time an inoffensive sludge. This modification of plain sedimentation is taken up in detail in Chapter X. Experience with it is so limited that the nature of the changes produced by aeration has not yet (1915) been satisfactorily explained.

In the early investigations made for the Massachusetts State Board of Health by Clark and Gage, satisfactory oxidation was not obtained until algæ developed on the surfaces in contact with the sewage, and vigorous growths of algæ were accompanied by more active nitrification than occurred with feeble growths. In later investigations with a tank containing plates of slate, aeration did not accomplish the best results until the slates were covered with a gelatinous film. In their report for 1913 these investigators attribute the great change in the sewage during aeration not only to oxidation but also to an apparent retention of the suspended matter and a large part of the colloidal matter by the gelatinous film. In the tank experiments by Dr. Bartow and others with activated sludge, such material is kept in suspension by the air and its effect as a pseudo-coagulant in adsorbing fine suspended and colloidal matter may prove an important feature of this treatment. The investigations show that such sludge is a very important element in the processes of clarification and oxidation.

CONTACT BEDS

The failure of various tank treatments to remove fine suspended and colloidal matter and the necessity of oxidizing the organic matter of tank effluents before their discharge into natural waters, led to the development of processes for transforming this organic matter into stable substances, the demands of which for oxygen are not great enough to exhaust the supply and cause putrefaction.

Perhaps the most logical step ahead from tank treatment is treatment in the contact or bacteria bed. This is a water-tight open tank filled with broken stone, cinders, coke, or other inert substances, commonly ½ to 1½ in. in size and about 4 ft. deep, Fig. 20. The material is often termed "ballast." When new the bed will have from 40 to 50 per cent.

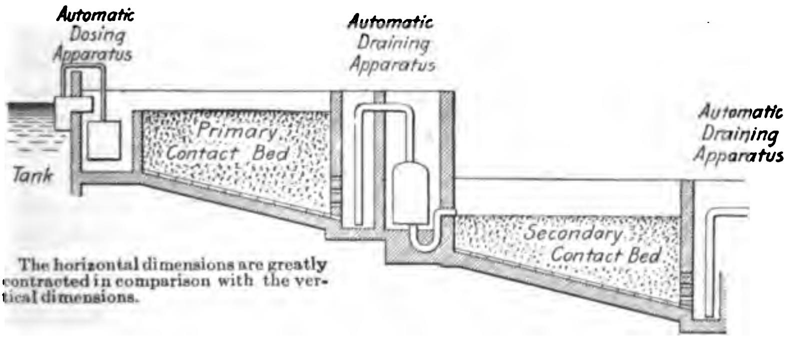


FIG. 20.—Arrangement of double contact beds.

of voids, but these gradually become filled with sewage solids, and the contact material has to be removed, cleaned and replaced after a service of perhaps 5 years. Such beds are often built in series of two or three; the effluent from the first or primary bed passes to the second and from the second to the third, being improved in quality by each successive treatment.

Contact beds are filled with sewage, allowed to stand full, emptied and allowed to rest. The cycle is then repeated. Schedules vary, according to the design and the rate of sewage flow, the time of resting depending upon the number of fillings. The following schedule may be assumed to illustrate such a cycle with three fillings per day:

Time of filling	1.0 hour
Time of contact	0.75 hour
Time of emptying	0.25 hour
Time of resting	6.0 hours
Time of cycle	8.0 hours

Physical and Biological Forces.—The work done by the contact bed depends upon two main forces, physical and biological.

While the sewage is commonly settled before being applied to contact beds, it still retains a portion of its settling solids and much suspended and colloidal matter. In the contact bed, the settling solids are deposited and largely retained in the interstices of the stone, and the colloids are withdrawn from the sewage by the physical attraction of the contact material. Certain sewage bacteria find a favorable habitat in the organic matter attached to the stones, and, in fact, doubtless increase its quantity by their rapid growth, thus forming a gelatinous film adhering to the contact material. Here the organisms thrive and convert the organic matter into more stable organic and mineral substances by oxidation, as described more fully in Chapter III, page 101.

Functions of the Cycle of Operation.—When starting a new bed the sewage is allowed to remain a short time in contact with the ballast during which time the suspended matter settles out on the surfaces of the contact material or is held mechanically by sharp points or in narrow passages. Much of the colloidal matter is thrown out of solution by coming into contact with the surfaces of the stones.

After the sewage is drawn off, much of the deposited matter is retained attached to the surface of the contact material, forming with the rapidly increasing organisms a film built up during successive cycles until each piece of contact material is coated by a slimy "bacterial jelly," generally believed to have the power of absorbing dissolved and colloidal substances. This jelly, being the result not only of the precipitation of solids but also of bacterial growth, requires a certain length of time and favorable conditions for its development, known as "maturing" or "ripening." During this time the biological equilibrium of the filter becomes established.

During the filling period, with a matured bed, sewage is applied in such quantity as to fill completely the voids in the ballast. Except in experimental work, beds are filled at one operation. The length of time required for filling does not appear to be very important and is usually dependent upon the size of the units and the frequency of filling them. It is undesirable to apply the sewage so quickly as to cause it to rush through the voids at a rate which will disturb the bacterial jelly and carry the accumulated solids toward the bottom of the bed, where conditions may be less favorable to their oxidation and may tend toward clogging.

During the second period, while the bed stands full of sewage, the suspended matter has further opportunity to become attached to the bacterial jelly, and the colloidal and dissolved matters come under influences having the same tendency. The decomposition begun in the

sewage while flowing through the sewers and going on within the bed during the previous period of rest and oxidation, is continued somewhat during the standing-full period, but as the supply of oxygen from the atmosphere and from the nitrates and nitrites is limited, the demand for oxygen soon exceeds the supply, after which oxidation gives way to putrefaction. If, therefore, the standing-full period is too long, putrefaction will be set up and go to such an extent that the filter cannot recuperate during the resting period, and the resulting effluent will be found inferior to that produced with a shorter contact period.

The third period is that devoted to drawing off the sewage after the filter has been standing full. As the liquid sinks, a fresh supply of air is drawn in to replace it, thus filling the interstices and furnishing a supply of oxygen from which the requirements of the living organisms can be satisfied.

The fourth period is the standing-empty or resting period. A new supply of oxygen is then furnished to the organisms and an opportunity given them to oxidize the organic matter previously retained in the bed. It is during this period that nitrites and nitrates are formed and the carbonaceous matter is oxidized, with the formation of carbon dioxide. In other words, the standing-empty period is devoted to a sort of wet combustion process. At first some believed that the changes which take place in the dissolved organic matter were due to direct action of bacteria during its passage through the bed, but Dunbar has shown experimentally that absorption plays an important part in the process. He says:

"If a solution of albumen, containing about as much organic matter as is present in ordinary domestic sewage, is placed in a sterile clinker filter, and the liquid examined every few minutes, it will be found that in this case a separation of the organic matter from the solution takes place. During the first few minutes 50 per cent. or more of the organic matter is removed, while later the action takes place much more slowly. The same action takes place, therefore, in the absence of bacteria as occurs with sewage in biological filters.

"This absorptive action may be easily demonstrated by the use of coloring matter, such as methylene blue. A deep blue solution of this coloring matter assumes a much lighter greenish color by simply being poured through a mature clinker filter; if the solution is allowed to stand for 2 hours in the filter it is almost completely decolorized.

"If sewage is colored with methylene blue, the blue color disappears in a day or two, but on shaking with air the blue color reappears, and these processes may be repeated for weeks. The solutions which have been decolorized in a clinker filter do not, however, behave in this manner; on shaking with air the blue color is not restored, because the coloring matter has not been reduced by the action of bacteria, as in the case of sewage, but has been retained by absorption in the filter. Experiments with fuchsin, litmus, and other similar coloring matters gave exactly the same results.

"By far the largest part of the purification which is effected while sewage is standing in a clinker filter is undoubtedly due to absorption. At the same time, to a certain extent, biological processes are occurring in the full filter. This may be concluded from the formation of carbon dioxide which takes place in the full filter. A sewage containing no free carbon dioxide, after being placed in a contact bed, contained 6.38 parts per 100,000 and 4½ hours later, 11.51 parts." ("Principles of Sewage Treatment," page 161.)

The difference between this theory and the Hampton doctrine was explained on page 21 of Chapter I.

The changes wrought in sewage are essentially biochemical. Organic nitrogen compounds are transformed, with the production of free ammonia, free ammonia is transformed into nitrites and nitrates, and nitrogen gas is evolved. Apparently nitrates are mainly formed while the bed is standing empty, and they are partly or wholly reduced by the sewage during the standing-full period, thus accounting for the relatively low nitrates in the effluent. The Royal Commission on Sewage Disposal concluded (Fifth Report, page 51) that knowledge of the action going on within contact beds is very incomplete, although the purifying agents seem to be worms, larvæ and insects as well as bacteria, but was unable to offer any information as to the respective amounts of work done by these several forms of life. Undoubtedly the proportion of work done by different organisms depends in part upon the character of the sewage, in part upon the method of operating the beds, and probably largely upon the biological balance within the bed. The resting period is apparently the more important phase of the cycle. It is believed that the ammonia is extracted from the sewage held in contact and is oxidized during the period of rest, the resulting nitrates and nitrites being diffused through the sewage subsequently applied to the bed. The Royal Commission on Sewage Disposal concluded that the withdrawal of suspended and colloidal matter from the sewage during its passage through the bed was not simply mechanical straining, for a matured unclogged contact bed will withdraw more suspended matter from the sewage than another bed similar in all other respects, but not matured. (Fifth Report, page 51.)

To sum up briefly the functions of the contact bed, it may be said to carry the work of the sedimentation tank a few steps further. It removes by surface attraction and absorption colloidal and dissolved substances, which are attacked, while held in or by the bacterial jelly, by bacteria and other organisms. The organic matter is thus converted into soluble stable organic substances, into mineral matter, into gases which either pass out of the filter dissolved in the effluent or are liberated into the atmosphere, and into humus-like solids, some of which are washed out of the bed; the remainder accumulates until the contact material must be removed, cleaned and replaced.

The Slate Bed.—This was devised by W. J. Dibdin about 1904, and consists of a water-tight tank filled with slates laid horizontally from 2 to 4 in. apart. The tank is filled with unsettled sewage, which is allowed to stand for a time to deposit its suspended solids on the slates. It is then drawn off slowly to avoid disturbing the sludge thus deposited, and the bed is given several hours' rest in which to regain its supply of oxygen from the atmosphere before being again filled. Manholes give access to the bed, which can be washed out by a hose stream and thus can be protected against undue accumulation of sludge. Dibden describes his work upon the slate bed as an attempt to make contact beds capable of continuous work by the application of the aerobic principle to the resolution of sludge into innocuous forms. The Royal Commission stated in its Fifth Report (page 67) that primary beds of this type should be regarded more as preliminary settling or septic tanks than as contact beds.

TRICKLING FILTERS

As the oxidizing power of the contact bed is dependent upon an ample supply of atmospheric oxygen absorbed mainly during the rest period, obviously the purifying capacity of the bed is limited by this period. Furthermore, less than one-half of the cubic contents of the new bed is available for the storage of sewage during the standing full period, and this space is gradually reduced by accumulations in the interstices of the ballast. The storage space is an important element governing the volume capacity of contact beds. These limitations encouraged efforts to devise beds which would accomplish similar work without being subject to such restrictions. The result was the trickling filter, similar in construction to the contact bed, except that it is not necessarily built within a water-tight tank. The filter material, usually about 1½ in. in size and from 5 to 10 ft. deep, is placed upon a draining floor, through which the water can flow freely. The sewage is applied to the surface of the filter as uniformly as possible by sprinklers or other devices. Fig. 21 is a view of the trickling filter at Fitchburg, Mass., built from the plans of D. A. Hartwell and the authors. It is usual to apply the sewage for a few minutes and then shut it off for a short time, as 5 minutes' dosing and 10 minutes' resting.

This construction offers as large an absorption surface as the contact bed, and while the sewage does not stand quiescent in the bed, it trickles very slowly over the stones and has ample opportunity to give up colloidal and dissolved substances to the bacterial jelly. The method of dosing is such that the bed may absorb oxygen continuously instead of during a brief period, as in the contact bed, and the danger of anaerobic conditions is greatly reduced. Experience has demonstrated that usually

the trickling filters are self-cleansing, while the material of contact beds must be removed and cleaned occasionally.

The theory of action is substantially the same in the two types of beds. The settling solids not removed by preliminary sedimentation are retained mechanically in the pores of the filter. The colloidal matter adheres to the surfaces through attraction and absorption, and the dissolved organic matter is absorbed by the bacterial jelly or acted upon directly, probably both processes occurring. The bacterial jelly coats the stones from top to bottom of a bed in mature condition and appears to perform its functions in relays, that at the top acting upon so much of the organic matter as its capacity will permit, that of the next stratum below attacking the substances passing by the upper one or perhaps emitted from it only partly oxidized, and so on to the bottom, where the oxidized sewage falls through the floor grating into the underdrainage system.

FIG. 21.—View of the sprinkling filter at Fitchburg, Mass.

The process of oxidation appears to be substantially the same as in the contact bed, the organic matter being converted into humus-like material or resolved into carbon dioxide and other soluble and gaseous products, and the nitrogenous substances being oxidized with the production of ammonia, nitrites and nitrates. Knowledge of the particular organisms present and of the work which each species does is lacking, but it is certain that the processes are chiefly aerobic, for failure of the air supply is quickly reflected in the inferior quality of the effluent. It is common to attribute the changes to bacteria, which undoubtedly should have credit for much of it, but the presence of vast numbers of higher forms of life, such as worms varying in size from those almost microscopic to earth worms 2 or 3 in. long, leads to the conviction that bacteria are not the only organisms laboring to transform the putrescible organic matter into substances less likely to cause offense and illness.

The periodic storage and disgorging of solids by the trickling filter is one of its most important and interesting functions. Being a community of innumerable living organisms, it is not surprising that it is affected by seasons and responds quickly to changes of temperature and other conditions. In summer, in the northeastern states, the quantity of solids in the effluent is about the same as in the applied sewage. Oxidation is then more active and nitrification will take place with doses not too great to permit efficient oxidation. A trickling filter designed and operated for that purpose will convert a large portion of the organic nitrogen into nitrates. In the fall, the efficiency of oxidation drops progressively with the lowering of the temperature, and the proportion of solids in the effluent to those in the sewage is also reduced. During the winter oxidation is low and much organic matter applied to the filter is stored in it. But with the first warm weather of spring, the filter emits great quantities of solids, far in excess of those in the sewage applied at the time. Thus the stored matter, including the bacterial jelly which has served its purpose, is ejected from the filter, which thus recovers its capacity. Likewise, the renewed activity of the organisms produces more complete oxidation, and the quality of the effluent gradually improves until it equals that of previous summers. Just what causes the rapid unloading of the filter is not known, although it has been suggested that the active movements of worms from one part of the filter to another may be responsible to some extent. The appearance of the accumulation in the filter, just before or while actively unloading, indicates a complete change in its character. The gelatinous films which covered the stones have become friable and resemble garden soil more closely than before.

Results.—The trickling filter is capable of converting putrescible settled sewage into stable effluent not subject to putrefaction under the most exacting conditions. The effluent contains much settling matter, especially during the spring unloading period. This should be removed from the effluent by sedimentation and the filter should be judged by the effluent thus settled. After sedimentation the effluent is usually more or less turbid and may be somewhat colored. There may be places where its quality would not be as good as the conditions required, and further purification or a substitute for trickling filters must be provided.

A practical operating difficulty not yet overcome is irregular distribution of the sewage over the surface. The sewage passes more quickly through those portions of the filter which are overdosed than through the underdosed portion. On this account, the effluent is composed of portions of the applied sewage which have received varying degrees of oxidation—some of which are perhaps little changed from their original condition and others of which are quite completely oxidized—rather

than of sewage all of which has been oxidized to the degree apparently represented by the analysis of the effluent.

No process thus far discussed will remove all bacteria from the applied sewage. Studies were made at Lawrence to determine the fate of spore-bearing disease germs, such as the anthrax bacillus, applied to filters. The conclusions are stated as follows:

"The results of these studies showed clearly that while there is a material reduction in the total number of bacteria and in the number of non-spore-bearing types in the passage of sewage through a septic tank, and through contact and trickling filters, there is no perceptible reduction in the number of spore-bearing bacteria. . . .

"From the results obtained, it is evident that sewage filters of coarse materials, operating at high rates, will not remove all the bacteria of the colon type. On the other hand, intermittent sand filters may remove all, or at least a large proportion, of these germs at certain times. . . . Both of the . . . . trickling filters constructed of coarse materials and operated with raw sewage at high rates showed a high removal, filter No. 135 removing over 90 per cent. and filter No. 137 about 97 per cent." (Report Mass. St. Bd. Health, 1904, page 233.)

The removal of bacteria at the Lawrence Experiment Station is probably considerably greater than that accomplished in practical sewage treatment plants. The following figures give the relative efficiencies of the methods of treatment at Lawrence, and are abridged from the report of the Massachusetts State Board of Health, 1908, pages 513–514.

	Removal of bacteria, per cent.
Single contact beds dosed with septic tank effluent....	53–55
Single contact beds dosed with settled sewage.........	58–62
Double contact beds dosed with settled sewage.......	77–78
Trickling filter[1] dosed with settled sewage.............	78–79
Trickling filter[2] dosed with settled sewage.............	96–97
Trickling filters dosed with settled sewage; effluent settled and refiltered through intermittent sand filter.	99
Intermittent sand filters............................	99

[1] This filter had 5 ft. of ¼- to 1-in. stone.
[2] This filter had 10 ft. of ¼- to 1-in. stone.

Neither contact beds nor trickling filters alone will remove all bacteria, although the trickling filters appear more efficient than contact beds. In spite of the turbidity and color of the effluent from trickling filters, its large bacterial content and considerable residual organic matter, this type of filter is now the most generally practicable means of artificial oxidation of sewage. Though incapable of producing as pure an effluent as some other methods, its efficiency is such as to render it suitable for adoption in many municipalities requiring artificial sewage oxidation plants.

Where it is necessary to carry oxidation to practical completion and to remove nearly all bacteria from the sewage, the contact bed and the trickling filter are not efficient enough to fulfill the requirements. Under such conditions their effluents must be submitted to secondary filtration or the more efficient purifying mechanism, the intermittent sand filter, must replace the contact bed or trickling filter. If higher efficiency is required simply for the removal of bacteria, this may be accomplished by disinfection.

SAND FILTERS

Intermittent sand filters are partly or wholly artificial. In the former case they are constructed by removing the loam and subsoil overlying

FIG. 22.—Intermittent filter with sewage distributor, Marlboro, Mass.

natural deposits of sand which are graded to provide a level surface. Where such sand deposits are not found, it is necessary to import sand. Such beds are usually from ¼ to 1 acre in area and surrounded by dikes. Fig. 22 is a view of a filter at Marlboro, Mass. It is important to dose a bed as quickly as practicable without disturbing the sand, so that the

sewage may be uniformly distributed; otherwise a large portion may penetrate the sand near the outlet pipes and thus cause overdosing of that portion and underdosing of areas at a distance. In some places suitable sand deposits have been found at a relatively high elevation above the ground water and underlaid with coarse sand or gravel, which provides natural underdrainage. Where these conditions do not obtain, it is necessary to lay underdrain pipes about 4 ft. below the surface of the bed to collect and carry away the effluent as rapidly as possible, thus preventing water-logging and aiding aeration.

Mechanical Straining.—The action of the sand filter may be best understood by considering separately its two functions, mechanical straining and transformation of dissolved organic substances into stable material. When sewage is applied to sand filters the coarser suspended particles are retained on the surface, forming a thin compact mat which does not allow water to pass readily. Where unsettled sewage is applied, such a mat will be formed in a relatively short time, and must be removed to enable the filter to do its proper amount of work. It has, therefore, come to be the custom to pass the sewage through sedimentation basins before applying it to filters, thus reducing the frequency of cleaning the beds. This, however, is accomplished somewhat at the expense of the porosity of the sand below the surface, for the fine particles in the settled sewage are capable of penetrating to a depth of 1 in. or more. Where unsettled sewage is applied, a portion of such matter is retained by the surface mat.

Sand filters, as ordinarily constructed, do not permit the suspended matter of the sewage to pass through the filtering medium and escape in the effluent. It is obvious, therefore, that the straining action of the filter may be approximately measured by the removal of the suspended matter, which, in the hypothetical sewage of page 200, amounts to 300 parts per 1,000,000, or 37.5 per cent. of the total solids on evaporation. While this statement is correct as applied to the first action of the filter, the organic matter retained in the upper portion of the filter is subject to biological action, and liquefaction and gasification are continually going on. Such action, however, is relatively slow, and while a substantial portion of the organic matter thus temporarily stored is finally oxidized and washed through the filter, the accumulation takes place much more rapidly than the oxidation and liquefaction. The effect of this accumulation in the upper portion of the bed is to reduce the effective size of the sand by filling the interstices with very finely divided particles, some of which swell and become gelatinous when wet. This condition prevents the rapid entrance of the sewage into the bed and likewise the free circulation of air. As an ample supply of oxygen is necessary, it is important that the sewage should penetrate the sand quickly after the full dose has been applied, and that the atmosphere should have free

access to the bed. Therefore it becomes necessary, as the accumulation of sewage matter increases, to remove from time to time the upper portion of the sand, thus restoring the bed to a condition approaching that which it possessed when new.

Oxidation.—The action of the filter as an oxidizing mechanism depends upon the life processes of bacteria. This has been clearly proved by the application of sterile sewage to sterilized filters, under which conditions the sand was incapable of transforming the organic matter into inorganic substances. This experiment also appears to disprove the old theory that the action of the filter is essentially one of direct oxidation by atmospheric oxygen contained in its pores. There may be some such direct action, and direct oxidation undoubtedly takes place with certain inorganic substances like ferrous sulphate, which is transformed into sulphuric acid and ferric oxide.

The biological action is apparently similar to that taking place in the trickling filter, but a much greater part of the work is done in the top part of the bed. Every particle of sewage comes into contact with the sand in such a way that a portion of the colloidal and dissolved matters may be thrown out of solution by attraction or absorption. These substances, together with the bacterial growths or zooglœa, attach themselves to the grains of sand in a gelatinous film covering each grain. This film, while appearing to absorb from the passing sewage dissolved and colloidal matter, is also the home of bacteria which feed upon and break down the complex absorbed organic matter and transform it into stable substances. It has been suggested that there is direct bacterial action upon the passing sewage. While this may be the case to some extent, by far the greater part of the work of the organisms is probably done after the organic matter has first been removed from the sewage by absorption. It is difficult to justify the theory of direct bacterial action after Dunbar has shown that sewage left a 3-ft. filter thoroughly purified within 10 minutes and shallower filters in shorter periods. ("Sewage Treatment," page 140.)

While in the process of oxidation the changes in sand filters appear similar to those in the trickling filter, there appears to be one practical difference. In the sand filter there is a much greater tendency toward mineralization and nitrification than is commonly observed in the trickling filter, although the latter, when constructed and operated for that purpose, is capable of carrying nitrification nearly as far as the sand filter. Under ordinary conditions, however, the work in the trickling filter is carried on at high speed, a common rate of filtration being 2,000,000 gal. per acre per day for a bed 7½ ft. deep. At this high rate, the tendency appears to be for the organisms to oxidize the organic matter only partially, converting it into stable organic matter rather than actually mineralizing it. On the other hand, the sand filter is operated at a rate

of perhaps 50,000 gal. per acre per day for a bed 4 ft. in depth, a rate fixed largely by the opportunity for reoxygenation of the bed. Obviously, this opportunity is very much less than in the case of the coarse broken stone or cinder filter. Sand filters have a marked avidity for oxygen, and if enclosed in an atmosphere of some other gas, as nitrogen, they will lose their power of oxidizing sewage. Dunbar maintains that this oxygen is absorbed in the gelatinous films surrounding the sand grains and explains the energetic oxidizing action as follows:

"We cannot regard the oxygen as being present in the ordinary molecular state, but must assume that it is ozoned by the high pressure existing in the gelatinous film and is thus rendered extremely active." ("Principles of Sewage Treatment," page 49.)

Air is drawn into the bed as the dose of sewage finds its way downward toward the outlet, and while there may be some circulation of air, particularly in the upper stratum, it is difficult to conceive that there is any rapid circulation through all parts of the bed except as it is drawn in according to Dunbar's theory. Therefore, the quantity of oxygen available for the oxidation of a dose of sewage is largely that which follows the dose into the pores of the filter.

These limitations, the small quantity of food due to the small dose of sewage and the small quantity of oxygen, determine the luxuriance of the growth of organisms. Therefore, the refinement in regulation necessary to maintain a proper balance of oxygen and bacterial life in the sand filter is such that the oxidation must of necessity be carried well toward completion, and conditions are particularly favorable for nitrification. It may be that one group of organisms produces the essential changes in the trickling filter and another group of organisms brings about the somewhat similar changes in the sand filter, and as the control of the trickling filter is refined to correspond more closely with that of the sand filter, those organisms best adapted to mineralization of the organic matter are permitted to develop more luxuriantly, and the work done by such a trickling filter more nearly approaches that done by the sand filter than is the case with trickling filters as ordinarily built and operated.

If the foregoing conception of the sand filter is correct, it must require a certain length of time after it is first put into operation in which to mature, which appears to be the case. During this time the films are forming about the sand grains and the bacterial life is becoming established and active. If new filters are started in the warm season, the "period of biological construction" may be as short as a week or 10 days, but if the filter is put into operation in the winter, it may be spring or early summer before it will become ripe and efficient, as indicated by the

presence of a substantial and increasing quantity of nitrates in the effluent.

An interesting conception of the sand filter, which has stood the test of time well, is as follows:

> "The filter becomes a delicate organism adapted to what is required of it, if its possibilities are not exceeded. The preparation required appears to be the introduction by the sewage of the particular organisms fitted to aid in this work, and their accumulation with a proper food supply, and other favorable conditions by which they become in time adapted to accomplish the most complete purification with the quantity of sewage received. Any change in quantity or mode of application may disorganize this working colony and prevent the best results until there is time for a readjustment adapted to the new conditions." (Rept. Mass. St. Bd. of Health, 1890, page 25.)

The formation of nitrates is an indication of the biological activity of sand filters. Nitrogenous matters are decomposed in filters either by the process of putrefaction or decay, the former taking place only when filters are overdosed or in some other way are prevented from receiving an adequate supply of oxygen. By these processes a part of the organic nitrogen is transformed into free ammonia. As pointed out in Chapter III, page 102, certain organisms are capable of converting free ammonia into nitrites and nitrates, and it is probably true that some transform organic matter directly into nitrites and nitrates. A portion of the nitrogen contained in the original sewage is converted into nitrogen gas, which escapes from the filter into the air.

Under certain conditions, filters which have been yielding effluents high in nitrates suddenly produce effluents high in free ammonia and occasionally in nitrites, and low in nitrates. Such a change accompanies overdosing, with consequent reduction in air supply, and the low temperatures of winter. The State Board of Health of Massachusetts found (Report, 1908, page 283) that the production of free ammonia and nitrites, as well as some nitrogen gas, under such conditions was sometimes due to the reduction of nitrates previously formed, the effluents in such cases being bright and non-putrescible.

As the primary function of the filter is to break down and oxidize the organic matter applied to it, and as by the several biological processes involved different nitrogen compounds are produced, much can be learned of the processes going on in the filter by ascertaining what nitrogen compounds are passed off with the effluent. High free ammonia in the effluent may be due to overdosing of the bed and the consequent reduction in the supply of air; hence, it may follow that because free ammonia is relatively high the bed is being overdosed or otherwise working at a disadvantage, and is producing an effluent containing compara-

tively large quantities of organic matter and one which may be putrescible. This conclusion should be reached with caution, however, as it may not be justified, because of the possibility that conditions are favorable to a reduction of the nitrates, as just explained.

If an effluent is high in nitrates and low in free ammonia, the evidence is conclusive that oxidation has been carried at least far enough to prevent putrefaction. The important influence of the seasons upon the quantities of nitrates and free ammonia in the effluent from a practical working sand filter plant are shown in Fig. 23, based upon averages of 229 analyses of the effluent from 33 filter beds.

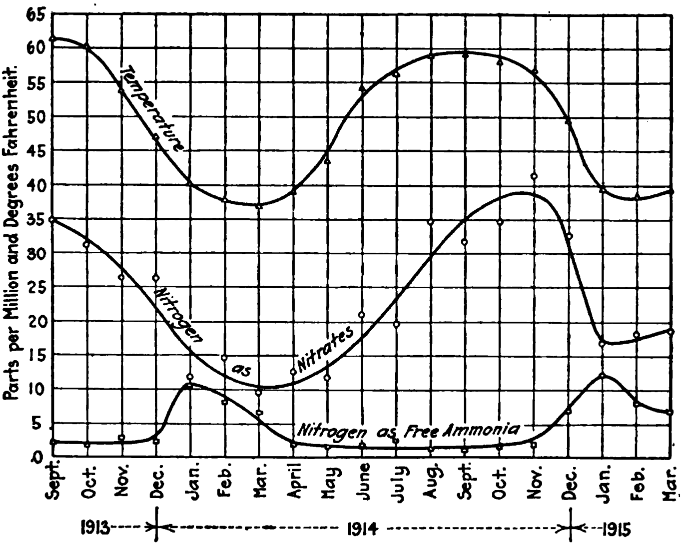


FIG. 23.—Effect of temperature on results of intermittent filtration.

Results.—The results obtained by the intermittent filtration of ordinary municipal sewage leave little to be desired. The effluent from a well-designed and carefully operated plant is usually practically clear, free from suspended matter, nearly colorless, without odor, containing little organic matter and but an extremely small part of the bacteria applied to the filter. It is not to be inferred, however, that the effluent from a practical working intermittent sand filter is suitable for domestic consumption. It is possible that at times some of the bacteria of the sewage pass through the sand bed, and if it is necessary to reduce the danger of the transmission of such organisms by the effluent from a sand filter plant, such effluent must be further treated by secondary filtration or by disinfection.

BROAD IRRIGATION

Broad irrigation is the term used by American engineers to designate the method of sewage disposal by which the sewage is allowed to flow over cultivated fields and percolate through the ground until it joins the natural ground water, incidentally watering and fertilizing the growing crops. Sewage contains chemical constituents which, when in proper form, possess fertilizing value, and the lure of financial return has made this method attractive to the popular mind.

Irrigation with sewage is carried out with two primary objects in view, proper disposal of sewage and the cultivation of crops from which a revenue may be obtained. The disposal of the sewage should be the primary object, in most cases, and the raising of crops secondary, controlled so as not to interfere with the sewage treatment. This has often been overlooked and the raising of crops and the deriving of revenue have been made primary considerations, with the result that the purification of the sewage has been neglected and in many cases little attempt to achieve this object has been made.

Application of Sewage to Land.—Sewage is applied to the land in many ways. In some cases it is allowed to flow over the land in thin films, gradually percolating into the soil until the liquid disappears at the lower edge of the field. In other cases it is allowed to flow in furrows or ditches a few feet apart, between which crops are planted. From the ditches the sewage percolates laterally and slowly, thus reaching and moistening the roots of the crops.

Sewage is sometimes applied to farms without preliminary treatment and in most cases without removing the suspended and colloidal solids, which are retained at or near the surface of the ground. They frequently accumulate in furrows or other depressions and form mats which do not allow the sewage to penetrate readily into the soil, thus exposing it for a considerable time to atmospheric conditions and permitting the escape of more or less objectionable odors. When plowed under, they experience the same slow change that takes place in the upper stratum of intermittent sand filters, previously described. These substances are thus added to the accumulation of humus in the top soil. All sewage contains grease and soapy materials which are particularly resistant to natural decomposition and tend to clog the pores of a filter. These fats are a serious detriment, as they prevent the full utilization of much of the fertilizing material applied to the land.

Sewage applied to farms operated to secure its efficient purification is oxidized by the same biological agencies as are active in the sand filter. The processes are aerobic, and it is therefore important that an ample supply of oxygen be provided. The biochemical changes which convert the organic matter into humus, carbon dioxide, ammonium compounds,

nitrites and nitrates change some of the fertilizing ingredients from substances which cannot be utilized by plants to compounds readily assimilable by them.

Influence of Character of Soil.—Of the sewage applied to a field, the greater part finds its way into the ground water and escapes with it, either through underdrains or through natural channels, into the neighboring streams. If the ground is flooded, the sewage as it recedes from the surface draws in behind it a fresh supply of air, which serves to maintain bacterial action as in the sand filters. The quantity of sewage which can thus pass through the land is obviously dependent upon the character of the soil. A stiff clay will allow very little water to pass, even if well underdrained, while a gravel or coarse sand will absorb a much greater quantity. An open sandy soil would appear to be much more favorable for sewage treatment, because of its greater capacity and the larger interstices for the retention of oxygen. Whatever the character of the underlying soil, however, the loam and subsoil so hinder the passage of water and the circulation of air that sewage farms can successfully treat but a small percentage of the quantity which can be treated on intermittent sand filters.

Action on Crops.—Sewage may act advantageously upon crops in two ways, by furnishing a required supply of water and by supplying fertilizing ingredients such as nitrogen, phosphates and potash. In arid regions, where any water for irrigation is in demand, the moisture derived from sewage may be valuable. In other places the natural rainfall is sufficient for the crops, although not always advantageously distributed. When there is a scarcity of water, sewage may be of value, but it is unfortunate that when there is the greatest quantity of sewage, during storms or when ground water is high, even with separate systems of sewers, the crops need no water and the application of sewage may be actually detrimental to them. The English custom of filtering some of the sewage delivered to farms, rather than using it for irrigation, is based on this fact.

Crops have the power of absorbing nitrogen compounds, phosphates, potash and other substances necessary to their growth. Some of these furnished by the sewage are not always in most available form, but must first be converted by bacteria into suitable forms for plant life. The benefits derived by the plants are not so great as might be expected, for the large quantity of water applied to the soil tends to leach it and carry away those substances which might be helpful to plant life. The effluents from many sewage farms are, for example, high in nitrates, a very good plant food, but the quantity of water passing through the soil apparently carries them away as rapidly as they are formed by the bacteria and before they can be assimilated by the plants. Therefore it seems that only a small portion of the fertilizing ingredients in sewage

can be utilized by the growing plants. Another consideration which affects materially the benefits derived from the fertilizing ingredients is the fact that in some climates plant life can take up such substances only during certain relatively small portions of the year. During the remainder of the year these ingredients are simply washed out of the soil by the sewage and serve no useful purpose.

For these reasons comparatively little benefit is derived by crops from the fertilizing elements of sewage applied to irrigation fields, and in most cases the supply of water has not proved of much value. The purifying ability of a sewage farm efficiently operated under favorable conditions is good, and the effluents from irrigation fields managed with a view to the purification of the sewage, even at the sacrifice of the crops if necessary, may be excellent in quality. Such effluents are sometimes high in bacteria and in many cases some microbes appear to pass through the ground.

DISINFECTION

In all the processes of treatment previously outlined, the purpose has been mainly to prevent sewage causing a nuisance. If it is desirable to reduce the danger of transmitting disease by sewage-contaminated water to a minimum, the bacteria in the effluent must be killed before it is discharged into the water. This can be done by disinfection, which is not difficult to carry out effectively under most conditions by methods described in Chapter XIX. The disinfectants have a powerful oxidizing effect and the organic matter forming the microscopic cells of the bacteria is attacked so vigorously that even spores are destroyed by small doses of some of the substances used.

A useful feature of disinfection which has probably not been fully appreciated yet is its availability as a measure for use when a treatment plant is put out of commission, wholly or in part, and the health of communities reached by the waters receiving the sewage would be jeopardized if the bacterial contamination caused by raw sewage were not greatly reduced. Disinfection of the sewage has been employed very satisfactorily in emergencies of this nature.

CHAPTER VII

SEWAGE DISPOSAL BY DILUTION

Disposal by dilution means the discharge of the sewage into a stream, tidal estuary, lake or sea. The theory of the resulting process of purification has changed from time to time, the current views being found on page 201 of the preceding chapter. Whatever it be, it may be fairly said today that, with suitable provision for screening and, if necessary, preliminary settling or sedimentation of the sewage, the process is one of the most economical, satisfactory and efficient methods of sewage disposal, and the one in most common use.

The extent of the use of dilution in 1915 in the United States, as estimated by the authors from data compiled by George W. Wisner, and in 1905, as estimated by George W. Fuller (*Trans. Am. Soc. C. E.*, vol. liv, Part E, page 148) may be summarized as follows:

	1905	1915
Population discharging raw sewage into the sea or tidal estuaries.	6,500,000	8,500,000
Population discharging raw sewage into inland streams or lakes.	20,400,000	26,400,000
Population connected to systems where sewage is treated in some way.	[1]1,100,000	6,900,000
Population connected with sewerage systems.	28,000,000	41,800,000

Oxygen Required to Oxidize Organic Sewage Matter.—The quantity of oxygen required to oxidize completely the organic matter in sewage varies greatly and little definite information concerning it is available. For illustrative purposes it may be assumed to average about 500[2] parts

[1] Some minor classes of treatment were not considered in making the estimate.

[2] Fuller estimates the absolute oxygen consumed by the permanganate test at four times that obtained by boiling 5 minutes ("Sewage Disposal," page 30). The average American sewages in Table 52 would, therefore, require from 236 to 532 p.p.m. or 1970 to 4440 lb. of oxygen per 1,000,000 gal. This "absolute" oxygen requirement does not equal that for complete oxidation. Adeney reports the total absorption of oxygen in 53 days by a sample of Belfast sewage shaken daily as 263.8 cc. per liter. (Fifth Report, Royal Commission on Sewage Disposal, Appendix 6, page 434.) This was equivalent to 2997 lb. per 1,000,000 gal.; the consumption of oxygen was incomplete but had become only a small amount daily. Lederer determined the biologic oxygen consumption of two grades of sewage at Chicago

per 1,000,000 or over 4000 lb. per 1,000,000 gal. of sewage. Fresh water, Table 5, contains at ordinary temperature and pressure about 10 parts per 1,000,000 of oxygen or 83 lb. per 1,000,000 gal. Thus it will be seen that it may require about 50 volumes of water for each volume of sewage in order to supply enough oxygen for complete oxidation, provided no re-aeration takes place.

In practice, the amount of oxygen required for complete oxidation is not so important as the rate at which the oxygen is absorbed. The complete process may take weeks or months, for the consumption is at a very low rate after the first few days. The object should be to dilute the sewage enough to enable re-aeration to supply the oxygen demands of the body of contaminated water or, more often, to prevent the diluted sewage from becoming putrid for a sufficient period to enable it to flow to larger bodies of water capable of affording practically unlimited dilution. Some idea of the great forces at work purifying polluted waters may be obtained from what is going on in New York Bay. The Metropolitan Sewerage Commission stated (1914 report, page 46) that in 1910 the estimated discharge of sewage into the bay was 765,000,000 gal. per day. If it be assumed that each million gallons requires 4000 lb. of oxygen, the total daily quantity required for complete oxidation would be 1530 tons.

Prof. W. E. Adeney estimated that the rate of absorption of oxygen by sewage matter in New York Bay would not exceed 0.75 cc.[1] per liter of water containing 0.5 per cent. sewage per 48 hours in the summer (1912 Report, Metropolitan Sewerage Commission, page 99); elsewhere in the same report (page 88) he stated that re-aeration of the water at the rate of 0.067 to 0.077 cc. per liter per hour (3.22 to 3.70 cc. per 48 hours) was about the maximum in inshore waters. The latter figures average about four and one-half times the former.

Prof. Earle B. Phelps held ("Discharge of Sewage into New York Harbor," 1911, page 147) that the oxygen obtained by re-aeration is almost negligible and amounts to not more than 0.045 lb. per 1,000,000 gal. or 0.0038 cc. per liter per hour at 12-ft. depth (page 62), but a small fraction of Adeney's estimates. The difference is due chiefly to different opinions regarding the rate of diffusion downward, particularly when water is stratified.

Re-aeration.—Re-aeration involves the factors of time, initial condition of the water, depth, and in brackish waters the element of stratifi-

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as 990 and 130 p.p.m. respectively (Industrial Wastes from Stock Yards and Packingtown, page 195). These figures are equivalent to 8266 and 1085 lb. per 1,000,000 gal. Dibdin estimated the oxygen consumed at one to three times the weight of the organic matter. (Water Supply and Irrigation Paper 185, page 16.) This would give from 3000 to 10,000 lb. of oxygen per 1,000,000 gal.

[1] To convert cubic centimeters per liter into parts per million, multiply by 1.43.

cation, as well as temperature of the water and the humidity of the air. The rate of re-aeration is exceedingly variable, even in different parts of the same body of water. Obviously it will take longer to saturate water to a great depth than to a lesser depth. The initial condition of saturation is important in that the greater the depletion of oxygen the greater will be the rate of re-aeration. Humidity, which affects the rate of evaporation and consequent density of the surface layers, has a bearing on the vertical circulation or "streaming" of the surface layers, which may be more or less affected in tidal estuaries by stratification.

Phelps, from a mathematical and experimental study of Fick's law of hydro-diffusion ("Discharge of Sewage into New York Harbor," page 89) has derived coefficients by which the amount of oxygen absorbed in a given time by a quiescent body of fresh water of stated depth and initial concentration of oxygen content may be determined. Phelps' figures were used by the Royal Commission on Sewage Disposal in an attempt to predict the amount of dilution required in order that the oxygen content of a stream will not fall below a prescribed amount under certain assumed conditions as to strength of sewage and depth and velocity of diluting stream. From many experiments the Commission divided streams into two classes, those moderately rapid, in which the water is assumed to be thoroughly mixed once an hour, and those in which the mixing takes place once in 6 hours, termed "very sluggish." (Eighth Report, page 10.) Taking the rates of re-aeration as calculated by Phelps for 1- and 6-hour periods, the Commission derived Table 55, on the assump-

TABLE 55.—SHOWING DILUTIONS OF SEWAGE LIQUORS WITH CLEAN RIVER WATER[1] REQUIRED TO PREVENT DEOXYGENATION BELOW 4 CC. PER LITER IN MODERATELY RAPID AND VERY SLUGGISH REACHES

(Royal Commission on Sewage Disposal, Eighth Report, page 11)

Depths of water in river (feet)	Time taken for river water to mix (hours)	Rate of re-aeration per hour expressed as percentage of saturation[2]	Dilution required for		
			Very good filter effluent taking up 1.0 p.p.m. dissolved oxygen in 24 hours	Ave. sewage taking up 130 p.p.m. dissolved oxygen in 24 hours	Strong sewage taking up 200 p.p.m. dissolved oxygen in 24 hours
2.5	1	0.99	0.40	55	85
	6	0.45	0.90	120	185
5.0	1	0.33	1.2	160	250
	6	0.15	2.5	360	550
10.0	1	0.12	3.0	450	700
	6	0.07	6.0	800	1200

[1] Clean river water is defined as that which will not absorb more than 0.2 cc. of oxygen per liter in 5 days.

[2] Saturation at 65°F. is taken as 6.63 cc. of oxygen per liter.

tion that the oxygen content should not be allowed to fall below 4 cc. per liter.

It should be remembered that all these calculations are based on experiments with fresh water and sewage stronger than many American sewages. Such calculations are quantitatively of small importance, although of interest in comparison with the results of similar degrees of dilution when the sewage is mixed with the water in other ways. Observations of the physical conditions of streams in the United States have indicated that when the ratio of sewage to diluting water was from 1 : 25 to 1 : 50, no nuisance will result. The Commission's requirements for dilution call for volumes of water about twice as large as American observations indicate are needed.

INFLUENCE OF SUSPENDED MATTER

Suspended matter has an important bearing on the dissolved oxygen content of polluted waters. That portion which forms sludge banks

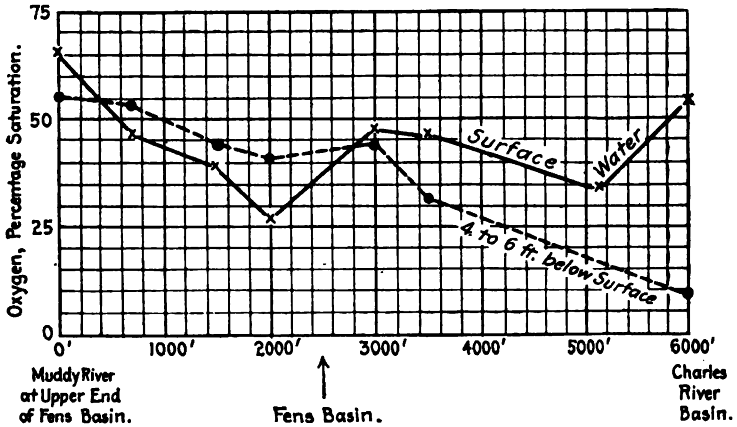


Fig. 24.—Effect of sludge deposits on oxygen content of Fens Basin.

on the bottom takes oxygen from the overlying water, so that, although the dilution is apparently sufficient, the continual abstraction of oxygen from the water by the banks may produce objectionable conditions. Fermenting sludge banks are considered responsible for a considerable reduction in the capacity of the Chicago Drainage Canal for preventing nuisance by dilution. (Report on Sewage Disposal, Wisner, 1911, page 14.) A striking example of the effect of sludge deposits was afforded by the condition of the Fenway Basin in Boston in 1902. This basin, formerly a tidal estuary, had its waters held at a certain elevation by a dam and tide gates. Muddy River and Stony Brook, two streams

draining a very thickly populated territory, discharged water which was practically weak sewage into the upper end of the basin. Very heavy sludge deposits formed in the basin, amounting in 1903 to about one-fourth of its cubic contents. H. W. Clark investigated the dissolved oxygen conditions in this basin and found marked stratification due to a mixture of salt and fresh water, interfering with vertical circulation, so that the underlying water was rapidly deprived of oxygen as it moved over the sludge (Fig. 24).

Relative Effects of Coarse and Fine Suspended Matter on the Oxygen Content of Water.—While it is important to remove settling solids, Dr. Arthur Lederer (*Jour. Am. Pub. Health Assn.*, 1912, page 97) and others have shown that the fine non-settling suspended matter is much more putrescible and sometimes more desirable to remove. Lederer's experiments indicated that while an average removal of 63 per cent. of the suspended matter by sedimentation increased the permissible amount of sewage which could be discharged into a stream by 31 per cent., without reducing the oxygen content below 30 per cent. saturation in 24 hours, the removal of the remaining 37 per cent. of suspended matter by filtration through filter paper resulted in an increase of 143 per cent. in the amount of permissible sewage when measured by the same standards.

Experiments described by Wisner and Pearse (Industrial Wastes from the Stock Yards, 1914, page 195) indicated that while reductions in the suspended matter by Dortmund, Emscher and chemical precipitation tanks, amounted to 66, 67, and 74 per cent. respectively, the reduction in the amount of biologic oxygen required amounted to only 32, 36, and 38 per cent.

AERATION OF SEWAGE-LADEN WATER

The forced aeration of sewage before its discharge into water, to increase its oxygen content, has been studied from many points of view. The earlier investigators decided that the amount of direct oxidation was very small. Phelps experimented with the aeration of New York sewage and concluded (Discharge of Sewage into New York Harbor, 1911, page 77) that this sewage, after a short period of septic action followed by aeration at the rate of 0.1 cu. ft. of air to 1 gal. of sewage, could be discharged into New York Harbor in two to three times larger quantities than were permissible with unaerated sewage. This estimate was based on the assumption that the oxygen content should not fall more than 20 per cent. in 6 hours, a half tidal period. The cost of installation of a plant for performing this object was placed at $2000 per 1,000,000 gal., with $2 per 1,000,000 gal. for power. Cost of labor and removing sludge from the septic tanks was not included in this estimate.

While aeration on this scale increases the period before putrefaction begins in sewage, the consumption of oxygen goes on long after the effect

of aeration is gone, for the amount of air introduced by such means is small compared with the amount necessary to render sewage non-putrescible indefinitely. Stability for 6 hours was considered all that was necessary at New York because tidal action was expected to carry the sewage into the open sea, with its abundance of dissolved oxygen. In the case of a slow-flowing river, where the sewage must be transported long distances, any effect of such re-aeration is likely to be soon lost. The effect of increasing pollution on the average annual dissolved oxygen content of the Merrimack River is shown in Fig. 25, compiled from reports of the Mass. State Board of Health. This reduction cannot be wholly attributed to increasing pollution, as the flow of water in the earlier years was much greater than later, thus giving greater dilution with less effect on the oxygen content.

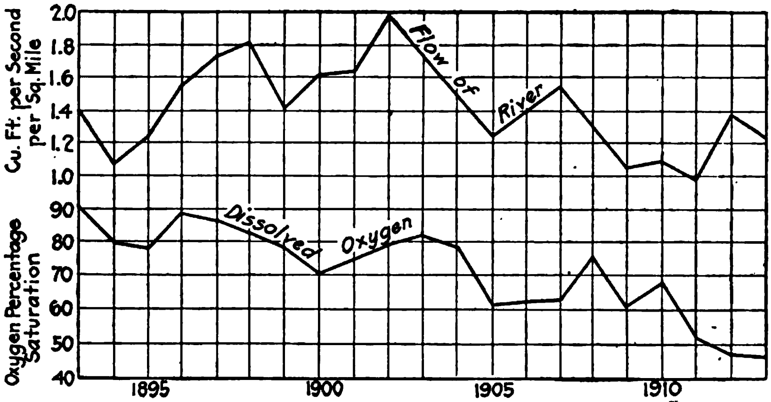


FIG. 25.—Effect of increasing pollution on dissolved oxygen content of Merrimack River.

Rate of Exhaustion of Dissolved Oxygen.—When sewage is discharged into water the dissolved oxygen content of the mixture is reduced in three ways: 1, by dilution; if sewage containing no dissolved oxygen is mixed with an equal quantity of fully saturated water, the oxygen content of the mixture is one-half that of the diluting water; 2, by the action of directly oxidizable substances, such as many of the products of the putrefactive process; 3, by the biological processes of bacteria.

The effect of the first is easy to determine when the relative volumes of sewage and water are known.

As to the relative importance of the second and third, Adeney concluded from his experiments with septic sewage that the easily oxidizable substances absorbed more oxygen in 1 day than did the indirectly oxidizable substances in 8 days. The amount of diluting water also has an effect on the rate of exhaustion of oxygen. Adeney found (Fifth

Report, Royal Commission on Sewage Disposal, Appendix 6, pages 72 and 80) that of two samples, one composed of 1 part sewage and 9 parts water, and the other 1 part sewage and 19 parts water, the latter, although one-half as strong as the former, consumed less than one-half as much oxygen in 3 days. This effect was confirmed by Lederer in experiments with Chicago sewage. (*Jour. Am. Pub. Health Assn.*, February, 1912, page 97.)

Many factors combine to make the depletion of oxygen greater in summer than winter. Not only does water contain less dissolved oxygen when saturated at the high summer temperature, but bacterial activity is then greatest. The flow of streams is then at a minimum, so that the ratio of sewage to diluting water is large. The effect of these

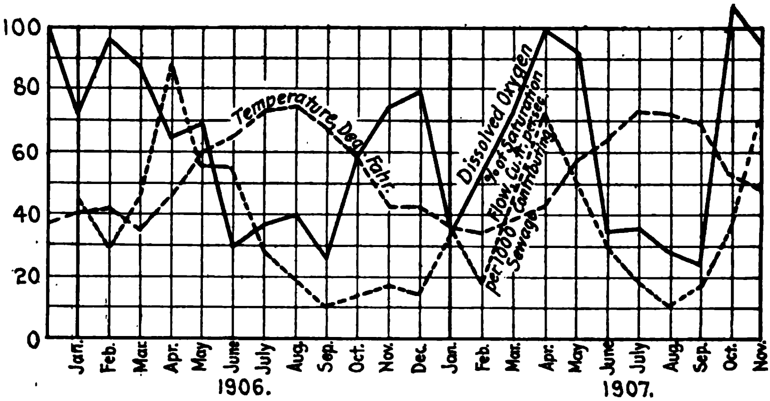


FIG. 26.—Relations of temperature, flow and dissolved oxygen in the Merrimack River.

influences in the Merrimack River is shown in Fig. 26, the discharge of the stream being based on reports of the United States Geological Survey and the analyses on the reports of the Massachusetts State Board of Health for 1906 and 1907. X. H. Goodnough estimated the amount of polluting matter in the Merrimack River above Lowell as equivalent to that produced by a population of 70,000, and the equivalent population discharging sewage below Lowell but above Lawrence at 182,300. These observations were taken in years of average run-off, and in dry years conditions must be worse. Conditions in the river have at times been such as to result in a legislative inquiry and special study in 1908 by the State Board of Health.

SELF-PURIFICATION OF NATURAL WATERS

Chicago Drainage Canal.—Probably the most extensive examination of the self-purification of an American stream was that of the Illinois

River in connection with the suit of the State of Missouri to restrain the State of Illinois and the Sanitary District of Chicago from discharging sewage diluted with water from Lake Michigan into tributaries of the Mississippi. The Chicago Drainage Canal was designed to carry a minimum of 3.3 cu. ft. per second of lake water per 1000 persons contributing sewage, as a result of studies by the Drainage and Water Supply Commission of 1887, consisting of Dr. Rudolph Hering, Chief Eng., Benezette Williams and Samuel G. Artingstall, then city engineer of Chicago. The preliminary figure suggested was 4 cu. ft. per second per 1000 persons, which was changed to 3.3 cu. ft. in the Sanitary District Act of 1889. The enormous quantity of trade wastes at present discharged into the canal was not foreseen, and the subject was con-

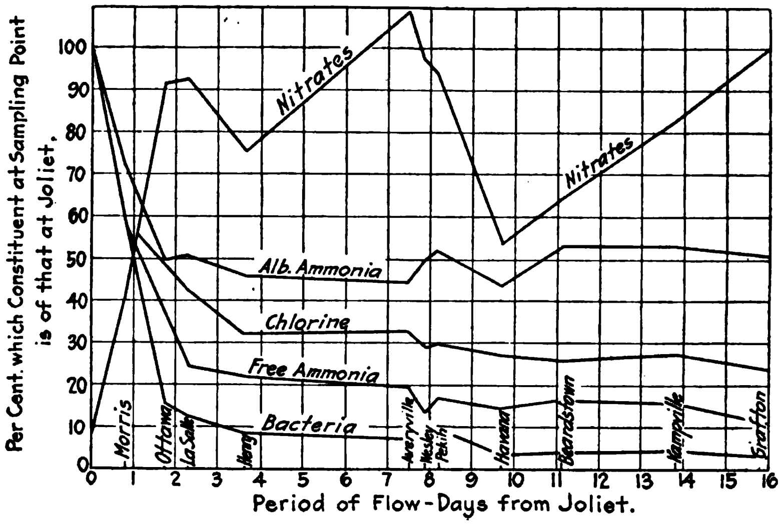


FIG. 27.—Self-purification of the Desplaines and Illinois Rivers. The percentage curve of the nitrates is based on the quantity at Grafton, instead of Joliet.

sidered from the standpoint of nuisance rather than the destruction of fish life upon which emphasis was laid by Black and Phelps in their report on New York Bay, which will be mentioned later.

In the Missouri-Illinois litigation, the plaintiff claimed that the operation of the drainage canal contaminated the water supply of St. Louis, while the defendants claimed that the processes of self-purification in the canal and rivers removed all danger of such contamination. The expert testimony was given by about 40 witnesses and was very contradictory. In general, while a very material self-purification was shown, Fig. 27, the Supreme Court held merely that the plaintiff's contention

was not proved but the case could be tried again, thus indicating that the possibility of pathogenic germs reaching St. Louis from Chicago was not disproved.

Fig. 27 is based on the testimony of Dr. E. O. Jordan and represents average conditions in January to June, 1900. The Joliet samples were collected below the junction of the Desplaines River, the Drainage Canal and the Illinois and Michigan Canal. For many years previous to the opening of the Drainage Canal, water had been pumped from the Chicago River at Bridgeport into the Illinois and Michigan Canal at an average rate of 600 cu. ft. per second. This water flowed through the canal 33 miles to the Desplaines River at Joliet. On January 17, 1900, the Chicago Drainage Canal was put in operation, and the use of the Illinois and Michigan Canal for sanitary purposes discontinued, only sufficient water for navigation purposes being pumped into it after that time.

At Wesley the effects of sewage from the city of Peoria, situated just above, were apparent. At Morris, about 1 day's flow below Joliet, the chlorine, free ammonia and bacteria dropped to only about 60 per cent. of their value at Joliet, while the albuminoid ammonia was reduced to 72 per cent. That this effect was almost entirely due to dilution is indicated from the following considerations:

Let d = parts per million of any constituent in water at Joliet
a = parts per million of same constituent in additional diluting water below Joliet
b = flow in cubic feet per second of river at Joliet
c = flow in cubic feet per second of river at Morris = 1.72 times flow at Joliet (testimony of J. A. Harmon).

If no influence but dilution were active the amount of any constituent at Morris, in parts per million, would be $[db + a(c - b)]/c$. If all the diluting water was similar to that of the Kankakee River, the principal tributary between Joliet and Morris, the following calculated analysis of water in the Illinois River at Morris, is obtained:

	Free ammonia	Albuminoid ammonia	Chlorine	Bacteria per cc.
Actual analysis at Morris..	2.46	0.60	24.5	445,000
Calculated analysis at Morris.	2.49	0.62	25.3	460,000

Wisner's report of 1911 on Chicago sewage disposal indicates that the dilution in summer is insufficient to prevent the reduction of the dissolved oxygen through the entire length of the canal to a point below 2½ parts per 1,000,000. In the lower 15 miles of the canal it averaged from 0 to 10 per cent. saturation, the latter figure being less than 1 part per 1,000,000 under summer conditions.

Illinois and Michigan Canal.—In 1888–1889 Prof. J. H. Long made for the Illinois State Board of Health an extended series of analyses of the Illinois and Michigan canal water to determine the amount of self-purification due to other causes than dilution and sedimentation. From Bridgeport at the head of the canal to Lockport, 29 miles below, the canal received no water other than rainfall and slight infiltration, and the constant passage of boats, together with the velocity of the water, prevented deposition. Dilution and sedimentation being eliminated, about 750 analyses were made to determine the purification effected by oxidation alone. A general summary of the results is given in Table 56, rearranged from "Sewage Disposal in the United States" by Rafter and Baker.

TABLE 56.—EFFECT OF OXIDATION ALONE IN ILLINOIS AND MICHIGAN CANAL

(Parts per million)

Place of collection	Bridgeport		Lockport	
Date of collection	May–Oct., 1888	Jan.–March, 1889	May–Oct., 1888	Jan.–March, 1889
Total solids	471.2	376.6	431.2	408.6
Suspended solids	129.2	27.2	69.8	24.6
Free ammonia	12.3	8.9	10.8	8.1
Albuminoid ammonia	2.6	2.8	2.0	2.5
Oxygen consumed	23.1	26.5	16.2	22.8
Chlorine	46.8	62.9	46.1	56.0

These results indicate that under the existing conditions self-purification by oxidation alone proceeded very slowly. The velocity of flow of the water is stated to have been 0.9 mile per hour, which gives a time period of 32 hours between Bridgeport and Lockport.

Massachusetts Studies.—The results of two studies of self-purification made in 1902 by the Massachusetts State Board of Health are given in Fig. 28. In its report for that year, the Board stated that the Sudbury River water above Saxonville was good. At this village it received the sewage of a considerable population and the liquid wastes from a woolen mill employing about 350 persons and washing 50,000 lb. of wool weekly. Here it became very foul and exceedingly offensive. In passing through extensive meadows it was improved so much that about 7 miles below Saxonville the pollution could not be detected by observation. The Charles River received at Milford the sewage of a population of about 2000 persons, and became exceedingly foul. In the next 5 miles of its course it flowed through a sparsely inhabited valley and at the end of this reach it was very much improved.

New York Harbor.—The investigations of the self-purification of the waters fouled by the sewage of New York, Fig. 29, were carried out in a comprehensive manner by the Metropolitan Sewerage Commission. The main results are given in Fig. 30, plotted from figures in the 1910 report of the Commission; this shows that in the main exits from upper New York Bay there is a gradual increase of dissolved oxygen as the ocean is approached. The figures are averages of the oxygen found at various depths and at various conditions of tidal currents. The percentage of saturation at the same time at different points in a cross-section of channel varied considerably. The samples were taken in the

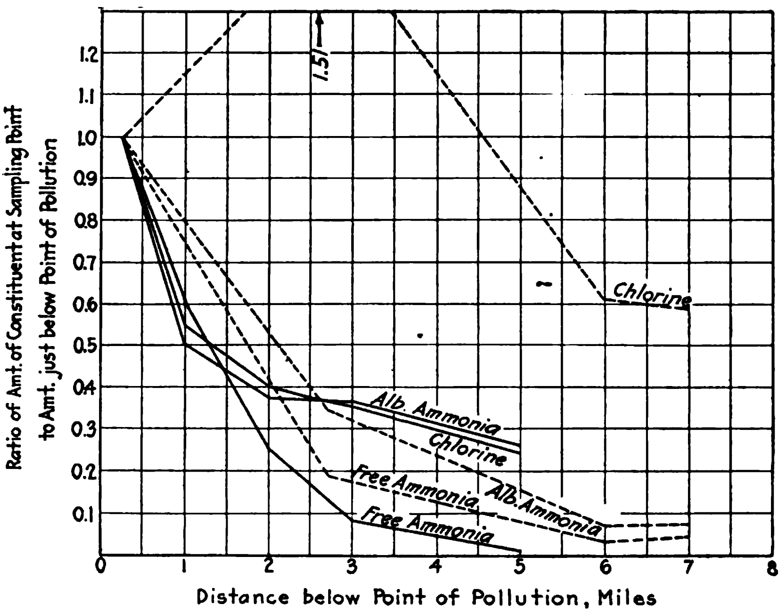


FIG. 28.—Self-purification of streams in Massachusetts. (Solid line, Charles River below Milford; broken line, Sudbury River below Saxonville.)

center of the current. In practically all cases, the waters of a flood current had more oxygen than those of the corresponding ebb current. In the Hudson River it was found that the percentage of saturation at the surface down to Fort Washington was 100 per cent. on July 12 and 13, 1911, but the average of deep samples was much less, as shown in Fig. 31.

English Streams.—The Royal Commission on Sewage Disposal investigated the condition of 27 streams receiving sewage and effluents from treatment works. Usually several examinations of the same stream were made when the amount of water flowing and the conse-

quent dilution were different. These experiments were conducted to determine the effects of polluting liquids in streams under various conditions, and analyses were not made for sufficient distances below the outfalls to demonstrate the completion of the purification by natural

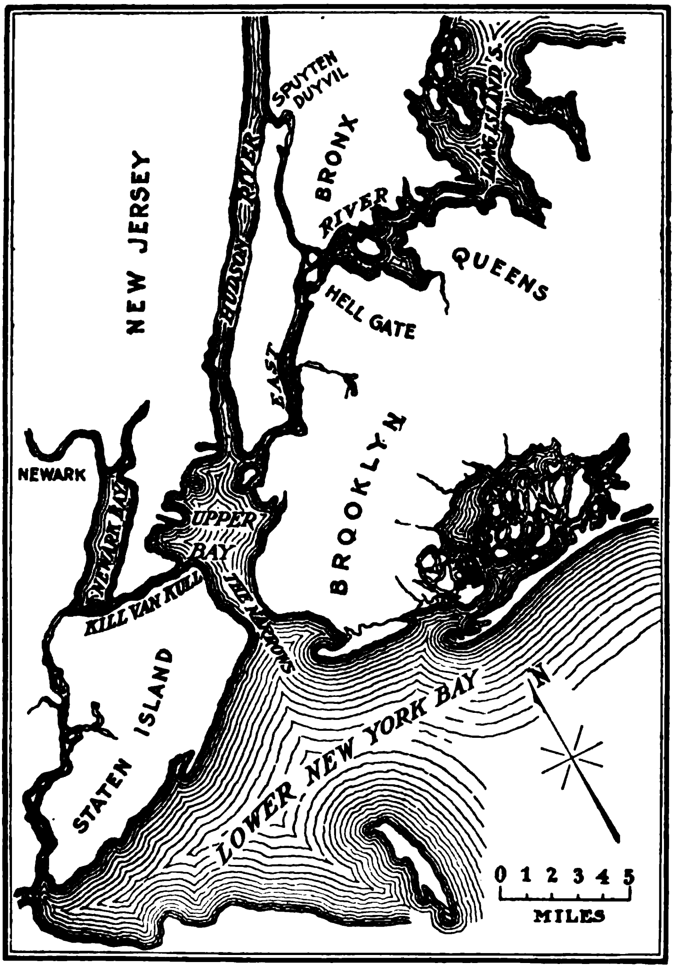


Fig. 29.—New York Harbor and adjacent waters.

agencies. In Fig. 32, for instance, the most distant analysis is only 32,100 ft. below the outfall, which distance, with an average velocity of 56 ft. per minute, is equivalent to only about 10 hours' flow, a period too short to demonstrate satisfactorily the action of forces other than sedi-

mentation and dilution. It will be observed that the percentage reduction in the different constituents is practically the same when the dilution is 20 : 1 as when 10 : 1.

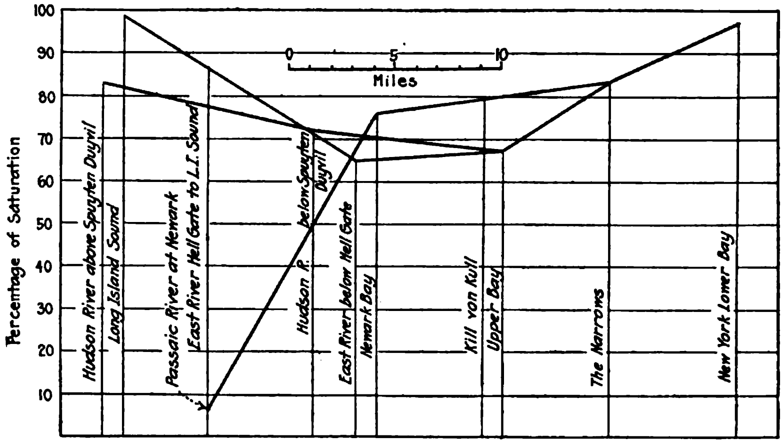


FIG. 30.—Dissolved oxygen in New York Harbor and adjacent waters.

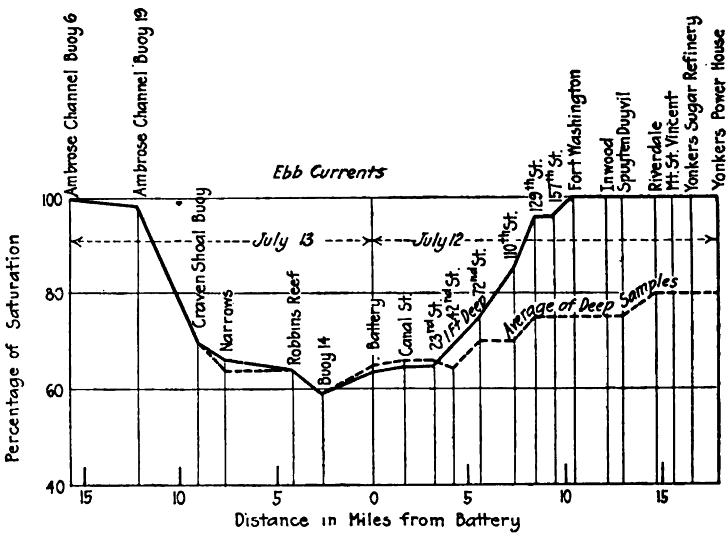


FIG. 31.—Dissolved oxygen in Hudson River and New York Bay.

The technical staff of the Commission gives in the appendix to the Eighth Report (page 64) the following instance as evidence of actual self-purification: Between Rugby and Coventry on the River Avon the

distance is 13 miles. The flow of the river in dry weather above Rugby amounts to about 4,500,000 imp. gal. per 24 hours. Above Coventry the flow amounts to about 7,500,000 imp. gal. At Rugby about 530,000 imp. gal. per day of effluent having the analysis given in Table 57 is discharged into the stream. The analyses of the river water in Table 57 show that the quality of the water at Coventry is as good or better than above Rugby, not including the addition of the Rugby effluent. The indication of its influence still remains in the chlorine.

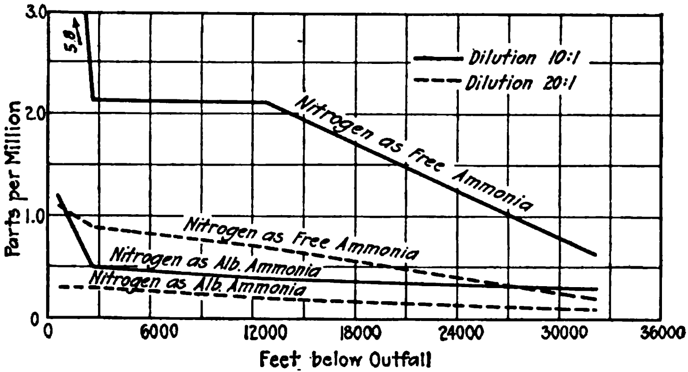


FIG. 32.—Self-purification in a small English stream after receiving the sewage effluent of a small town.

TABLE 57.—ANALYSES SHOWING RESULTS OF DISCHARGE OF RUGBY SEWAGE WORKS EFFLUENT ON RIVER AVON

(Royal Commission on Sewage Disposal, Eighth Report, Appendix, page 64)
(Parts per million)

	Rugby effluent	River water	
		Above Rugby	13 miles below Rugby
Nitrogen as			
Free ammonia	21.2	0.50	0.10
Albuminoid ammonia	2.0	0.40	0.30
Nitrates	5.6	1.50	2.20
Oxygen absorbed[1]	14.3	3.10	2.50
Dissolved oxygen absorbed[2]	19.7	2.50	2.30
Chlorine	94.5	24.20	41.00

[1] From N/8 permanganate in 4 hours at 27°C.
[2] In 5 days at 18°C.

During 1907 and 1908 the Royal Commission conducted experiments in which filter and tank effluents flowed through wooden channels 150

to 360 ft. long. The object was not to study self-purification, but to observe the effects of effluents of different characters on the channels in which they were flowing. Analyses taken of the liquid at the entrance and exit of the channel showed in many cases a considerable degree of purification, as in the following example.

At Chorley an experiment begun in August, 1907, ended in September, 1908. The channel was 150 ft. long and the flow through it continuous. The channel became coated with a gray growth near the upper end and a green growth in the lower half, and contained considerable black mud and brown suspended matter. In spite of the more or less unfavorable appearance of the channel, analyses indicated a considerable improvement in the character of the liquid after traversing the length of the channel, as shown in Table 58.

TABLE 58.—EFFECT OF FLOW IN WOODEN CHANNEL 150 FT. LONG, ON CHORLEY SEWAGE WORKS EFFLUENT

(Royal Commission on Sewage Disposal, Eighth Report, Appendix, page 194)
(Parts per million)

	Influent to channel	Effluent from channel	Improvement, percentage
Nitrogen as			
Free ammonia	24.0	22.9	5
Albuminoid ammonia	1.4	1.2	14
Nitrites	1.2	0.8	....
Nitrates	6.7	9.9	48
Oxygen consumed[1]	16.0	9.8	39
Dissolved oxygen absorbed[2]	21.0	9.6	54

[1] In 4 hours at 27°C. from N/8 permanganate. [2] In 5 days at 18°C.

ENGINEERING STUDIES OF SELF-PURIFICATION

In addition to the results of chemical and bacterial studies, there are records of the effect of the discharge of known amounts of sewage into waters, prepared mainly by the investigations mentioned on page 29.

Stearns' Investigation.—After an investigation for the Massachusetts State Board of Health, Frederic P. Stearns, then its Chief Engineer, concluded (Special Report on Water, 1890, page 791) that if the flow is less than 2½ cu. ft. per 1000 inhabitants an offense would be almost sure to arise.[1] With larger volumes than 7 cu. ft. per 1000 inhabitants, the pollution would be too small to cause any nuisance. Where the water is to be used for manufacturing purposes, the amount of dilution should

[1] Stearns estimated that each person contributed to sewage daily an average of 0.015 lb. free ammonia, 0.003 lb. albuminoid ammonia, 0.218 lb. dissolved solids and 0.042 lb. chlorine.

be greater, he stated, and in the case of a stream used for domestic water supply he was unwilling to say that any degree of dilution would make the water entirely safe to use.

Goodnough's Investigations.—In 1902 another investigation was made for the Massachusetts State Board of Health by Goodnough, who succeeded Stearns as Chief Engineer of the Board. He narrowed the range of dilution fixed by Stearns and summarized the results (report St. Bd. Health, 1902, page 452) as showing that where the quantity of water available for the dilution of the sewage in a stream exceeds about 6 cu. ft. per second per 1000 persons contributing sewage, objectionable conditions are unlikely to result. Where sewage was discharged at many outlets into a large body of water he thought that objectionable conditions might not result from somewhat less dilution than that named. In every case where the flow was less than 3.5 cu. ft. per second per 1000 persons objectionable conditions had resulted.

In 1902, in a letter to the Committee on Charles River Dam, Goodnough made the following statement:

"The degree of dilution which has been found necessary to prevent unsanitary conditions where sewage is discharged into a stream, assuming 75 gal. of sewer per person, ranges between 20 to 1 and 60 to 1. In estimating the degree of dilution of the sewage, no account has been taken of the purifying effect of the water of the basin itself." (Evidence, etc., before Committee, page 108.)

J. Herbert Shedd testified in 1902 before the Committee on Charles River Dam that about 5 cu. ft. per second of the ordinary flow of the river, per 1000 persons contributing sewage, would render the presence of the sewage unobjectionable (Evidence, etc., before Committee, page 365).

In 1908 and 1913 Goodnough made for the Board investigations of the condition of the Merrimack River, the results appearing in special reports. The condition of other rivers was examined to obtain corroborative evidence regarding pollution, and it was found that wherever the dilution was 3.4 cu. ft. per second or less per 1000 persons, a nuisance followed. Where serious pollution was observed with higher rates of dilution the nuisance was usually due to the discharge of large quantities of industrial wastes into the stream. No case of dilution was found between 5.8 cu. ft., where industrial wastes and sewage combined to cause offense at Webster, and 9.2 cu. ft. at Ware, where the effect of the sewage was noticeable for a considerable distance, except at cities along the Merrimack. The dilution in the latter river was 8.7 cu. ft. below Lawrence and 7.6 cu. ft. below Haverhill. Below Lawrence the surface of the river was reported to be covered often for a long distance with froth, scum, and oily and greasy matters, particularly in summer, when

there was a noticeably disagreeable odor at times along the south bank. Except at the latter place, the pollution had not rendered the river offensive except near some of the large sewer outlets and where banks of sludge were exposed at low water, the report stated, and these conditions could be ameliorated by improving the methods of discharging sewage into the stream, so as to provide uniform diffusion through the river water.

Pollution of Ohio Rivers.—In 1897 the Ohio State Board of Health had an investigation of the condition of certain Ohio rivers made under the direction of Allen Hazen. In discussing the results he stated that in the case of sluggish streams, or of streams the waters of which are already somewhat polluted, the quantity required for proper dilution may become 6, 8 or even 10 cu. ft. per second per 1000 population. (Preliminary Report of an Investigation of Rivers, page 32.)

New York Harbor.—The Metropolitan Sewerage Commission of New York estimated in its 1910 report that the sewage discharged into New York Harbor was diluted with 32 parts of water, and that this ratio would become 1 to 13 about 1940. The flow of diluting water in the harbor was estimated at 4.7 cu. ft. per second per 1000 population, which would be reduced to 2.65 cu. ft. in 1940.

The permissible limit of pollution in sea water depends upon whether the object in setting the limit is the prevention of nuisances or the protection of fish. This has been emphasized in disputes over the permissible pollution of New York Bay, where the dissolved oxygen should not be reduced below 70 per cent. saturation, according to Black and Phelps, in order that food fishes might continue to live in the waters. Fuller stated on this topic:

"With respect to guarding against objectionable odors I think it is clearly necessary for the chemists and bacteriologists to keep in mind that putrefaction does not exist so long as oxygen remains at all. In fact you can go further than that, and say that so long as oxygen is available from nitrates, nitrites, or other oxidized salts there is substantially no putrefaction. I am aware that that does not provide for one feature that may be of some importance, and that is the protection of major fish life. I believe, however, that the European custom in many places is sound in indicating that 30 per cent. of the dissolved oxygen necessary for saturation provides a reasonable margin in the case of a majority of species of fish life of the larger kinds. Perhaps some may call for more, but so long as there is 30 per cent. remaining at all places at all times, it is a matter of deduction from our well-established laws of biology and chemistry that there can be no putrefaction. The larger number of the principal rivers in this country serving as public water supplies do not contain as much as 70 per cent. of the oxygen necessary for saturation. Among the rivers with which I have been personally familiar through analysis, I may mention the Merrimack River at Lawrence, Mass. Twenty years ago it had as

low as 50 per cent. and sometimes but 30 per cent. of dissolved oxygen. In those days it served as the water supply for Lawrence without being filtered, and in the last 17 years, since filtering, it has been regarded as one of the good water supplies of the world. I believe that if this 70 per cent. margin suggested by Dr. Soper were applied to Lawrence, it would show that the Merrimack River at that place was not providing a proper disposal for the sewage at Lowell and the cities above, notwithstanding the fact that it provides an excellent water supply at that point." (*Trans. Am. Inst. Chem. Engrs.*, vol. iii, page 392.)

CRITICAL POINT BETWEEN STABILITY AND PUTREFACTION

Rideal has deduced an equation for finding the critical point at which the amount of organic matter in water becomes so great that all oxygen is absorbed and a condition of putrefaction begins. ("Sewage," 1900, page 16.) This formula is:

$$XO = C(M - N)S$$

in which X is the flow of the stream in hectoliters per minute,

O, the grams of free oxygen in 1 hectoliter,

S, the hectoliters of sewage or effluent discharged per minute,

M, the grams of oxygen required to consume the organic matter in 1 hectoliter of sewage or effluent, as determined by the permanganate test with 4 hours' boiling, and deducting the nitrite and nitrate oxygen,

N, the oxygen available in nitrites and nitrates, and

C, the ratio between the amount of oxygen in the stream and that required to oxidize the organic matter in the sewage or effluent.

Phelps made the following comment on this formula:

"This formula serves to distinguish three possible cases. If C be negative, the effluent not only will not putrefy by itself, but by virtue of its excess of available oxygen, will tend to improve the condition of the stream, if the latter be already polluted. If C be greater than unity, the effluent will draw upon the oxidizing power of the stream, diminishing the power of the latter to deal successfully with further pollution; but in this case the stream will not itself become foul from this effluent. If C be positive, but less than unity, the stream will be overburdened by the addition of the effluent, and will become foul." (*Technology Quarterly*, vol. xviii, 1905, page 127.)

Hazen has deduced an equation for the same purpose, in "American Civil Engineers' Pocket Book" (first edition, page 982). It has the form $D = x/s = fm/o$ in which

x is the volume of water,

s, the volume of sewage,

o, the amount of dissolved oxygen in the water,
m, the result of the "oxygen consumed" test, expressed in parts per million, and
f, a factor depending upon the method used for determining the oxygen consumed, and which is approximately 4 for the 5-minute test as made in this country, 6 for the 2-minute test, 7 for the 4-hour test as made in England, and 12 for the 15-minute test as made in England.

These equations can be considered of value only for illustrating the principles involved, as there are too many uncertain and changeable factors which greatly modify the results. Oxygen consumed by the permanganate test bears no constant relation to the dissolved oxygen consumed by bacterial processes. Furthermore, no account is taken of the re-aeration of the water of the stream receiving sewage. As mentioned on page 68, the Royal Commission on Sewage Disposal has attempted by taking these factors into account to predict the dilution required in rapid and sluggish streams of different depths, by sewages taking up specified quantities of dissolved oxygen in 5 days. Hoover and MacGuire have also made experiments along similar lines (*Engineering News*, May 28, 1914) with a view to determining the relations existing between tests for dissolved oxygen in the laboratory and those taking place after discharge of an effluent into a stream.

CONDITIONS FOR SUCCESSFUL DISPOSAL BY DILUTION

Some conditions favorable to successful disposal of sewage by dilution are: (*a*) freshness of the sewage; (b) freedom of sewage from floating matter and solids capable of settling, a condition attained by treatment; (*c*) thorough diffusion through the diluting water; (*d*) diluting water of high oxygen content; (*e*) swift currents to carry the sewage to points of unlimited dilution; (*f*) biological equilibrium; (*g*) absence of slips and coves tending to facilitate sedimentation accompanied by sludge deposits.

The stipulation entered into between the United States Government and the Passaic Valley Sewerage Commission of New Jersey regarding the discharge of sewage into New York Bay is in part as follows:

"It is stipulated and agreed that the sewage, waste and other matter passing through the said trunk sewer shall first pass through coarse screens to remove therefrom all large floating matter, and after passing through such coarse screens shall pass through a grit basin or basins where the heavy matter therein shall be taken out as far as practicable, from which basin or basins the sewage and other matter shall pass through self-cleansing mechanical screens having clear openings of not over 0.4 in.

"As the sewage comes from the fine screens, it shall also pass through

sedimentation basins. The sewage after passing through said grit basin and said self-cleansing mechanical screens shall enter the sedimentation basins or settling tanks, consisting of a number of units, each approximately 225 ft. long and 15 ft. deep. Each tank will have a normal capacity of not less than 1,250,000 gal., making an aggregate tank capacity sufficient to meet the requirements as stated herein. The tank capacity shall always be such as to provide a detention period of not less than 1 hour at the maximum rate of flow of the sewage and a detention period of the daily average flow of such sewage of not less than 1½ hours. The mean lineal velocities through said tanks shall not be over 0.5 in. per second for average flow, and 0.75 in. per second for the maximum flow. In addition to and in connection with these basins scum boards shall be provided to retain the floating matter, and proper and adequate devices shall be used to remove the retained scum and deposits from the settling basins.

"The sewage and waste thus screened and settled is then to flow into a pump well, whence it is to be pumped under pressure through a tunnel to a point in the New York Bay near Robbins Reef Light, at which point it is agreed that the matter passing through the said tunnel shall be dispersed into the waters of the New York Bay through a series of outlets discharging 40 ft. or more beneath the surface of the water at mean low tide. From the end of the tunnel, connections shall be made with 4 or more discharge pipes extending across the current, spaced about 100 ft. apart, laid in trenches on the bottom of the Bay, and of a size decreasing in diameter from about 6 ft. to 2 ft. On the top of these discharge pipes will be a series of not less than 150 tees of a diameter not exceeding 1 ft., spaced approximately 10 ft. apart. On each of these vertical tees shall be placed outlets arranged to discharge horizontally across the tidal current, and the extent of the dispersion area used for this system of outlet pipes shall cover at least 3.5 acres of the bottom of the bay.

"The Passaic Valley Sewerage Commissioners further agree with the United States that in the operation of said sewer system at all times the following results shall be secured, either through compliance with the requirements of the immediately preceding paragraphs or through requisite lawful additional arrangements, viz.:

"There will be absence in the New York Bay of visible suspended particles coming from the Passaic Valley sewage.

"There will be absence of deposits objectionable to the Secretary of War of the United States in the New York Bay coming from the Passaic Valley sewage.

"There will be absence in the New York Bay and its vicinity of odors due to the putrefaction of organic matters contained in the Passaic Valley sewage thus discharged.

"There will be a practical absence on the surface of New York Bay of any grease or color due to the discharge of the Passaic Valley sewage at the dispersion area or elsewhere.

"There will be no injury to the public health which will be occasioned by the discharge from the said sewer into the Bay of New York in the manner proposed and no public or private nuisance will be created thereby.

"The absence of injurious effect from said sewage discharge, upon the property of the United States situated in the Harbor of New York.

"The absence of reduction in the dissolved oxygen contents of the waters of New York Bay, resulting from the discharge of Passaic Valley sewage, to such an extent as to interfere with major fish life."

Authorities are agreed that there is very little evidence of direct injury to public health resulting from serious pollution by sewage of harbors and lakes not used as sources of water supply. Nevertheless, it cannot be doubted that sound public policy dictates the desirability of maintaining as high a standard in this direction as may be practicable.

BASIC INFORMATION FOR PLANNING DILUTION PROJECTS

In order to pass intelligently upon the disposal of large quantities of sewage in fresh water, (*a*) hydrographic surveys may be required; (*b*) a study of the quality and temperature of the lake water and of the currents and of the winds affecting them may be needed, with chemical and bacterial surveys of the water at different points and depths, from which can be plotted zones of pollution of different degrees; (*c*) a study of possible locations for and forms of the sewer outlet may be desirable to ascertain any danger of pollution of the water supply, of nuisance by washing of the sewage back to the shores of the lake, or of objectionable sleek or grease upon the water surface; and finally, (*d*) the engineer will have to weigh carefully the character and extent of preliminary treatment required, the desirability of disinfecting the sewage or effluent before discharging it into the lake, and the relative necessity for treatment of the sewage or purification of the water supply. The influence of the chemical composition of the sewage upon the amount of dilution needed has already been explained in this and previous chapters.

Tides.—If a river discharges into a tidal estuary, a mixture of fresh and salt water is produced. If the estuary is short and steep, it clears itself with each ebb-tide; if long and with complex entries, the water may oscillate backward and forward with a varying degree of salinity. The salt water, being heavier than the fresh, follows the bottom and on the flood tide the underrun of salt water results. The velocity of the ebb-tide is greater than that of the flood-tide because the flow of fresh water is in the same direction as that of the outgoing salt water, instead of in opposite direction, as on the flood tide. This subject will be taken up later in this chapter.

Effect of Wind.—The wind blowing over the surface of any body of water tends to set up currents in it, owing to its frictional resistance. On-shore and off-shore winds produce complementary currents in opposite directions at greater depths, as described later in this chapter. If

there is an on-shore movement of the water at the surface, there must be a corresponding off-shore movement beneath the surface. If the currents be along the shore, the entire movement may be in one direction.

On Lake Michigan the travel of the surface currents is about 5 per cent. of the wind travel and they may extend to a depth of from 30 to 40 ft. (1909 Report, Lake Michigan Water Commission). Major W. V. Judson reports observed surface current velocities of 2.3 ft. per second and probable velocities of 4.4 ft. W. H. Wheeler states in "Practical

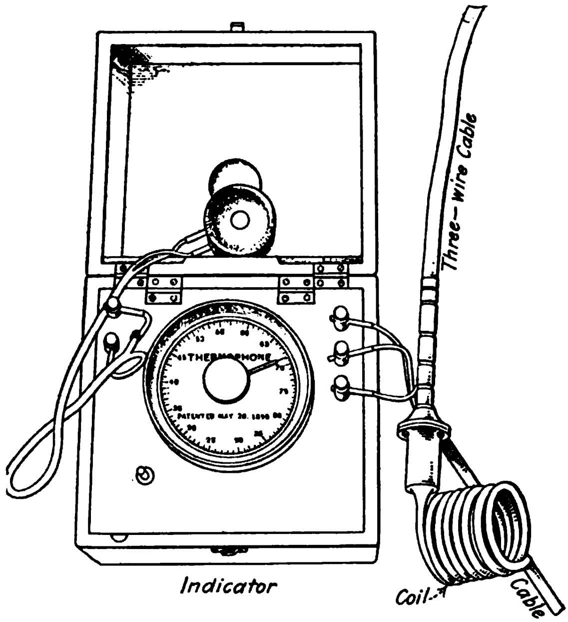


FIG. 33.—The thermophone. (From Whipple's "Microscopy of Drinking Water.")

Manual of Tides and Waves" that continuous winds from one direction in the Gulf of St. Lawrence produced an appreciable velocity of flow at a depth of 30 ft. In the North Atlantic ocean a study of many observations by the United States Hydrographic Service led to the conclusion that the set of the surface currents amounted to about 3.2 per cent. of the wind velocity. (*Monthly Weather Review*, 1902.) Somewhat similar figures were obtained by Prof. George C. Whipple in Lake Erie, where the currents due to wind ranged from 3 to 6 per cent. of the wind travel. (Report on Cleveland Water Supply, 1905.)

Knowledge of the currents produced by the wind has been gained largely by studies of temperature changes in the water at different

depths. For making the temperature readings, the most useful instrument is the thermophone, invented by Warren and Whipple. It is an electrical thermometer, Fig. 33, consisting of a sensitive metallic coil, which may be lowered to any desired depth and an indicator connected with the coil by lead wires. Electric current is supplied by dry batteries and a telephone receiver is connected to the indicator by which the operator can detect, by a buzzing sound, when the movable pointer is made to approach the correct temperature reading, at which the buzzing ceases. Thus by reading the dial at this point, the operator may ascertain the temperature of the distant coil. Thermophones, adjusted for a range from 40° to 250°F. will give readings correct to 0.2°. A description of the details of this instrument may be found in Whipple's "Microscopy of Drinking Water," page 88.

The vertical circulation in the Charles River Basin, Boston, due to winds and waves, is described in *Engineering News*, March 10, 1910, by M. F. Sanborn. The basin contained a mixture of fresh and salt water at the time of the investigation, and the effect of winds was determined by observing the vertical distribution of chlorine through the water. The greatest depth at which complete mixing of the water was caused in this way was 5 ft. with a wind velocity of 5 miles per hour, 10 ft. with a wind of 7 miles, 15 ft. with a wind of 9½ miles, and 20 ft. with a wind of 14 miles. Sanborn doubted if the same results could have been obtained at other seasons of the year, owing to different proportions of fresh and salt water, and he did not consider them applicable to a lake of similar dimensions containing fresh water only.

Studies at Duluth, Milwaukee and other places indicate that the effect of wave action may extend to a depth of at least 40 ft. (1909 Report, Lake Michigan Water Commission.)

If the depth at which the bottom changes from mud or clay to sand is a criterion of the depth of influence of wave action, the influence reaches a depth of 55 to 60 ft. at Duluth, 40 to 45 ft. at Chicago and Milwaukee, and 33 to 38 ft. at Cleveland.

Effect of Temperature.—Temperature changes in the water cause vertical currents due to differences in its specific gravity. As the surface water cools in the autumn it becomes heavier and sinks, thus causing currents reaching to increasing depths until all the water acquires the temperature of maximum density, 39.2°F. If it were not for the wind, circulation would then cease, but the slight differences in specific gravity near this temperature facilitate action by the wind so that the whole body of water probably reaches a somewhat lower temperature than 39.2°. In the spring, when the icy surface water becomes heavier as it is warmed, vertical currents are again established and continued until the temperature of the lower strata has reached a point somewhat higher than 39.2°, due to wind action, as previously noted. The sub-

ject has been investigated by Desmond FitzGerald, who pointed out the different effects of temperature and winds in deep and shallow lakes in *Trans. Am. Soc. C. E.*, 1885, vol. xxxiv, page 74.

Seiche.—Besides the influences of wind and temperature there are certain movements in large bodies of enclosed water caused by local changes in barometric pressure and by winds blowing for a long time in one direction, known as seiches. After the immediate cause is removed, the body of water tends to oscillate in a rhythmic period which, according to observations made in Scotland, may be represented by the following formula,

$$t = 2l/3600\sqrt{dg}$$

where t is the time of oscillation in hours, l the length of the lake or its width in the case of a transverse seiche, d is the mean depth in feet of the lake along the line of oscillation, and g is the acceleration of gravitation.

This formula has been found by Whipple to agree very well with observed conditions on Lake Erie during storms. (Report on Cleveland Water Supply, 1905.)

TIDES

In tidal waters the influence of winds and temperature is very often completely masked by the powerful currents and the circulation induced by tidal changes. The mean range of the tidal rise and fall is 9.6 ft. at Boston, 4.4 ft. at New York, 1.2 ft. at Baltimore, 5.2 ft. at Charleston, 6.5 ft. at Savannah, 1.2 ft. at Key West, 0.5 ft. at Galveston, 3.9 ft. at San Diego, 4.2 ft. at San Francisco, 6.4 ft. at Astoria, and 7.7 ft. at Seattle. It has been estimated that a difference of 1 in. in the barometric column will cause over a foot difference in the elevation of the surface of the sea. Strong wind may pile the water up in front of it upon the nearest shore, or may cause unusually low water.

At the turn of the tide the incoming salt water follows the bottom, owing to its greater density, while the overlying brackish water is still traveling seaward. Gradually, as the depth of the incoming wave increases, the motion of the entire section changes, flowing inward with increasing velocity until the maximum velocity, or "strength of the tide," is reached. The velocity again tapers off until ebb-tide begins, when the whole mass of water flows outward with increasing velocity until the maximum run is reached. The velocity then decreases until the condition first mentioned again prevails and a new cycle begins.

The tidal prism of a tidal basin is the volume of water contained within it between the limits of high and low water.

The study of currents in tidal waters is complicated by the necessity of considering the inertia of the mass of moving water, the configura-

tion of the bottom and the frictional resistances. Where there are several inlets from the ocean to a tidal basin, as in New York Harbor, the conditions are sometimes very complicated. Inasmuch as these tidal currents must be relied upon to remove sewage, the engineer should endeavor to acquire a comprehensive knowledge of them before locating the outlet of a sewerage system, in order to be sure that offensive conditions will not arise at any stage of the tide.

For the study of the very pronounced currents induced by tidal action, delicate chemical and bacterial analyses are often of slight

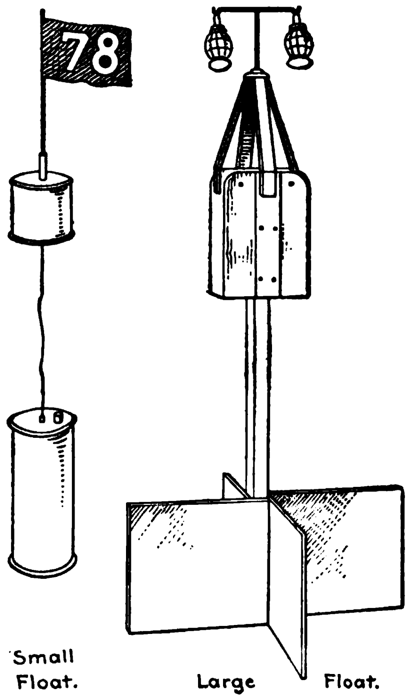


Fig. 34.—Types of floats used by Metropolitan Sewerage Commission.

significance as compared with observations on floats placed in the water near the proposed outfall.

The New York Sewerage Commission, in its observations of currents, used floats of the types shown in Fig. 34 and described as follows in its report for 1910:

"*Can Floats.*—The first consisted of tin cylinders, an upper and a lower one connected by a wire. The upper cylinder was 5⅜ in. in diameter by 5 in. in length; it was empty and sealed and carried a small red flag on

a staff set in a socket on the upper end of the cylinder. From this upper can, which in action was partly submerged, a larger can 6¼ in. in diameter by 14 in. in length was suspended by a copper wire of such length as to permit the larger can to float in the current whose velocity was to be determined. This larger can was weighted with sand until the top of the upper can was nearly level with the surface of the water. This type of float had the advantage of ease of handling, ease of preparation for use, a small area exposed to wind and small cost. On the other hand, where traffic was congested—as in the East River—they were destroyed by the paddles and propellers of steamers. When required for night work they were unable to carry a lantern.

"*Spar Floats.*—A second type consisted of a stick of timber, 2 × 2-in. by 5-ft., buoyed by a cork float at the top and carrying four vanes of sheet iron, 12 × 24 in. in size. The vanes were nailed to the stick and stayed by a wire which connected their outer edges. This float was readily made and proved to be effective in use. Its chief defect was that the plates were too easily bent when the float was out of water.

"A third type was like the second except that it was more substantial. It was made of a 3 × 3-in. by 6-ft. stick buoyed at one end by being built up to 12 × 12 in. for 24 in. from the top and weighted at the other by four vanes of No. 14 gage iron 18 × 21 in. in size, secured by bolts. A ⅝-in. rod projected about 4 ft. above the top and was provided with two arms, from which were suspended red and white lanterns at night. As this float was heavy and difficult to handle, and as the rod was easily bent, a light stiffening frame of ⅛ × 2-in. iron was attached to the head of the float and supported the rod just below the lantern. To this frame was welded a hook to be grappled in removing the float from the water. This design proved satisfactory for the rough seas experienced in December, 1909, in the Lower Bay and among the tugs, car floats and ferries of the East River."

These floats were followed day and night and their position determined at frequent intervals by means of a sextant or by estimating the bearing from some known point.

Salinity Observations.—The measurement of the salinity of the water in a somewhat polluted tidal basin sometimes affords important information concerning the possibility of discharging more sewage into it. For instance, the tidal currents in the East River at New York, (Fig. 30), have been reported by the U. S. Coast and Geodetic Survey to have maximum, average and minimum velocities of 7.8, 5.0 and 2.7 ft. per second. It was long held that in the tidal movements there was a resultant southerly flow from Long Island Sound to New York Upper Bay amounting to about 11 per cent. of the northerly flow. The subject was reviewed for the Metropolitan Sewerage Commission of New York by O. H. Tittman, Superintendent of the Coast Survey, who decided that it was difficult to detect a net flow in either direction, and if there was an excess toward the south it could not exceed 1 per cent.

Float observations by the Commission confirmed this opinion that there was no resultant flow. Observations of the salinity of the water were finally made which established the fact that there is such a flow, and consequently non-settling sewage matter discharged into the East River will eventually be carried out to sea, if not removed in some other way earlier.

This method of studying tidal flow is based upon the fact that while the normal specific gravity of sea water, which depends upon its salinity, varies from about 1.022 to 1.028, it is practically constant near any one place in the ocean. Off New York, it is about 1.025, corresponding to a chlorine content of 18,000 parts per 1,000,000 under the assumption that 88.6 per cent. of the total excess specific gravity above unity is due to chlorides. In this case, a sample of harbor water having a specific gravity of 1.015 must be a mixture of 60 per cent. of sea water and 40 per cent. fresh water.

A description of the Commission's methods has been given by Kenneth Allen in a paper on "Use of the Salinometer in Studies of Sewage Disposal by Dilution." (*Jour. Assoc. Eng. Socs.*, April, 1911.) The instrument is 12 in. long, with a stem $\frac{5}{32}$ in. in diameter, carrying a 4-in. scale reading specific gravities from 1.00 to 1.03 by intervals of 0.0005. A sample of water is placed in a tall cylindrical glass and the salinometer lowered into it. Both the thermometer and hydrometer scales are read and the hydrometric reading is corrected by an amount, ranging with the temperature from − 0.0011 at 35°F. to + 0.0028 at 82°F., which was determined by experiment.

The salinity of tidal waters has a direct effect on the vertical currents of the sewage discharged into them unless they are warm and fresh, when the sewage will probably remain near the bottom. According to Allen, experiments with colored water or sewage discharged below the surface in waters of different densities and at different depths did not always give harmonious results. In general, where the injected liquid had a specific gravity from 0.004 to 0.016 less than that of the harbor water, and the depth of discharge was between 20 and 40 ft., its rate of ascent was from 0.10 to 0.17 ft. per second. This rate is about one-third that of varnished wooden balls, carefully fitted up to have a specific gravity of unity, which were released at different depths below the surface.

If the volumes of water moving in and out of a tidal basin on the flood and ebb tides are known, and also the volume of upland water that passes out on each tide, the new sea water that enters during each flood tide to assist the upland water in replenishing the supply of dissolved oxygen required to aid in its purification can be computed. Hazen's formulas for this purpose (*Jour. Assoc. Eng. Socs.*, June, 1906) are as follows:

If E is the volume of ebb tide passing any place,
R, the volume of river water in the ebb tide,
S, the average proportion of sea water in ebb tide, then
$E\ (1 - S)$ is the river water passing out at one ebb, and
$R/E(1 - S)$, the proportion of river water that passes out and does not return.

Using these formulas, Allen has computed that under normal conditions about 17 per cent. of the water coming into New York Upper Bay was fresh sea water, and the proportion entering the Bay in dry weather was nearly 13 per cent.

EFFECT OF CHARACTER OF WATER RECEIVING SEWAGE

Quiet and Running Water.—The absorption of oxygen by quiet water is very gradual, but when the surface is agitated, diffusion takes place more rapidly. In this respect a running stream tends to absorb oxygen somewhat more rapidly than a pond. Many other factors which play an active part in self-purification, such as sedimentation, wind and sun, bacterial and plant life, are more active in quiet waters than in running streams.[1] Prof. Wm. T. Sedgwick says:

"It was for a long time believed, even by engineers and sanitarians, that running water purifies itself. The facts upon which this thesis rested were the obvious disappearance of gross pollution, introduced at a given point in a stream, at points below, and chemical evidence that water drawn at such lower points was less polluted than at the place of contamination . . . . Very early, however, it began to be discovered that the purification, which was so obvious on inspection and was further demonstrated by chemical analysis, was, from the sanitary point of view, incomplete and insufficient. As long ago as 1874, the Rivers Pollution Commissioners of the British government, after careful investigation concluded that 'there is no river in the United Kingdom long enough to secure the oxidation and destruction of any sewage which may be discharged into it, even at its source.' From that day until the present it has gradually become more and more clear that such purification as takes place is largely, if not almost wholly, a purification by dilution, and that many of the dangerous elements, especially micro-organisms, once admitted, are not in fact removed, but

[1] In Whipple's report on the quality of the Cleveland water supply the following statement is made concerning the effects of sedimentation and sunlight on lake water: "Sedimentation is a potent factor in the self-purification of lakes. During periods when the water is quiet much suspended matter settles to the bottom. If this happens at a time when the lower layers are stagnant, this suspended matter may not be again brought into circulation for many months, and during this time many of the typhoid fever bacilli may die a natural death in an unfavorable environment. There is always a chance, however, that the sediment may be again stirred up by vertical currents and the germs be again scattered through the water. . . . Sunlight exerts a powerful germicidal action on the bacteria in those layers of water which are near the surface but the sun's rays lose their energy very rapidly below the surface." (Report of 1905, page 70.)

only scattered in a running stream or river. At the same time, it has also gradually become plain that sedimentation and the destruction of micro-organisms by various agencies are more completely effected in standing than in moving water; so that modern sanitary science has reversed the tenet of 30 years ago and now unhesitatingly affirms that it is quiet water rather than running water that 'purifies itself.'" (Report of Pittsburgh Filtration Commission, 1899, page 17.)

Goodnough has stated from experience with ponds at Easthampton, Attleboro and elsewhere, that sewage discharged into a pond or slow-moving stream, such as the Charles River Basin, has a less noticeable effect than an equal volume of sewage has upon a rapidly moving stream of equal volume. He further said:

"In connection with public water supplies, the advantages of the storage of polluted water in large reservoirs in the removal of the effect of pollution have been recognized for many years, and the available evidence furnished by the observations of the effects of the discharge of sewage into ponds in the State indicates that, whatever effect the sewage discharged into the proposed basin may have upon its waters, the effect is likely to be less than it would be in the case of the discharge of an equal quantity of sewage into a flowing stream receiving the same quantity of water." (Report on Charles River Dam, page 311.)

The same view was expressed by Clark as follows:

"There is a certain popular belief that running water purifies itself more quickly than still water; the fact is, however, that, with oxygen present in the still water and as good conditions for proper bacterial growths, the still water purification is at least as energetic as the purification occurring in running water." (Report on Charles River Dam, 1903, page 291.)

Notwithstanding the recognition of the importance of sedimentation and of the action of organisms in relatively quiet water, the great value of aeration of water by wave action, the agitation of propellers and the fall over dams and riffles, must not be ignored. It is easily conceivable that a stream may be so seriously polluted as to become putrid if practically quiescent, whereas, if caused to fall over dams at frequent intervals along its course, it will absorb sufficient oxygen to enable it to maintain the processes of oxidation and thus avoid putrefaction.

Fresh and Salt Water.—When sewage is discharged into salt water there is a greater tendency than under similar conditions in fresh water for the sewage solids to form sludge banks. This tendency is caused partly by chemical action of the salt water on the sewage and partly by the inability of the salt water to carry as much matter in suspension as the fresh water.

Sea water normally contains about 20 per cent. less dissolved oxygen than distilled water, so that, other conditions being equal, fresh water

is able to dispose of more sewage than sea water. Adeney concluded from experiments that salt water, when depleted of its oxygen, reabsorbed oxygen from the air nearly three times more quickly than did distilled water. He attributed this result to a certain "streaming effect" by which the saturated surface water was carried down. (Royal Commission on Sewage Disposal, Fifth Report, Appendix 6, page 2.) Phelps found that this streaming effect was due to certain conditions of the experiment. Arguing from other considerations relating to the degree of solubility of oxygen in salt water and the viscosity of the latter, he concluded that salt water would not absorb oxygen more rapidly than fresh water, but probably much less rapidly. (Discharge of Sewage into New York Harbor, 1911.)

Clark conducted experiments with bottles of polluted sea and distilled water from which he concluded that the oxygen in the salt water mixture was absorbed very much more rapidly than in the fresh water mixture. He also decided from the results of these tests that:

"1. The greater number of bacteria in the supernatant liquid were found under fresh water conditions.

"2. The greater bacterial growth in the muds occurred under fresh water conditions.

"3. The greater relative number of anaerobic growths occurred under salt water conditions, both in the mud and in the supernatant liquids.

"4. In the salt water experiments the number of bacteria which, when the water was plated, would grow in hydrogen, *i.e.*, under anaerobic conditions—exceeded in number in some instances those which would grow in air—*i.e.*, under aerobic conditions." (Report on Charles River Dam, 1903, page 285.)

Adeney and Letts, from independent studies, came to the conclusion that there was no great difference between the relative behavior of the two kinds of water. (Fifth Report, Royal Commission on Sewage Disposal, 1908, Appendix 6; also seventh report, 1911.)

About all that can be done at the present time, in view of the conflicting testimony, is to call attention to the established facts: first, that salt water contains about 20 per cent. less oxygen than fresh water; second, that salt water exercises a marked influence to precipitate sewage solids and thus form putrefying sludge banks, which rob the overlying water of its oxygen. It would seem, therefore, that, other conditions being equal, so far as the production of actual nuisance is concerned, disposal into fresh water is the safer of the two.

Nuisances from Aquatic Growths.—Prof. E. A. Letts has studied the nuisance caused by decomposition and subsequent decay of certain green seaweeds, notably Ulva latissima, in the upper reaches of the Belfast Lough. (Fifth Report, Royal Commission on Sewage Disposal, Appendix 6.) During the summer and autumn, in windy weather,

the Ulva is washed ashore in enormous quantities, forming banks frequently 2 to 3 ft. thick which extend at times for miles along the coast. They rapidly putrefy in warm weather and create an intolerable odor, "the stench at low tide being often quite overpowering, and the air heavily charged with sulphureted hydrogen." He attributed the growth to the discharge of sewage into the tidal estuary, and suggested treatment of the sewage in contact filters to obviate the evil. The Commission, in a later report, questioned the importance of the pollution in furthering the growth of this weed, and stated that the growth would probably still persist if all the sewage were removed.

Upon the subject of troublesome growths Dr. Gilbert J. Fowler has said:

"It seems, therefore, as if sea water inhibits to some extent the growth of algæ, etc., which develop by utilization of the organic matter. There would thus appear to be actually less organic matter present after long exposure of mixtures of sludge and sea water than after similar mixtures with tap water are exposed. A comparison of losses on ignition after the exposure with those of the original samples as given in an earlier table also shows that little or no actual loss of weight has occurred in either case. Too much stress should not, however, be laid on this point, as the quantities to be weighed were comparatively small.

"No doubt under natural conditions some of the organic matter would be consumed by infusoria and other low forms of life and thus enter once more upon a cycle of change. The inorganic matters will, however, eventually deposit, carrying with them also some of the organic matters of a more resistant kind." (Fifth Report, Royal Commission on Sewage Disposal, Appendix 6, page 557.)

Whipple, after studying the relation between the number of organisms present in waters and the chemical analyses of the waters, concludes that an excess of chlorine, which is an indication of pollution, slightly increases the number of organisms, and that free ammonia and nitrates are particularly influential in determining the amount of microscopic life present, but he cautions against treating cause for effect and states that free ammonia, instead of representing food for organisms, may be due to their decay. The extent to which one organism lives on the products of decay of another is not well known. ("Microscopy of Drinking Water," pages 143, 146.)

SEWAGE DILUTION IN BOSTON HARBOR

The location of sewer outfalls in Boston Harbor, both those of the Main Drainage Works and the Metropolitan Sewerage Districts, was determined with the aid of float experiments. These cases are instructive, as the effect of the sewage after the outfalls had been constructed has been investigated from time to time.

The first of these outlets constructed was at Moon Island, Fig. 35. It served a population of about 377,000 in 1912. Preliminary to the selection of this location, 50 free-float tests were made upon currents in the vicinity of Moon, Castle, Thompson's and Spectacle Islands. This outfall was designed for a discharge only on the ebb tide. The float observations indicated that suitable ebb currents passed both Castle and Moon Islands, but on account of certain factors not relating to the dilution question, Moon Island was adopted. Floats leaving the vicinity of that island on the early ebb tide traveled seaward at an

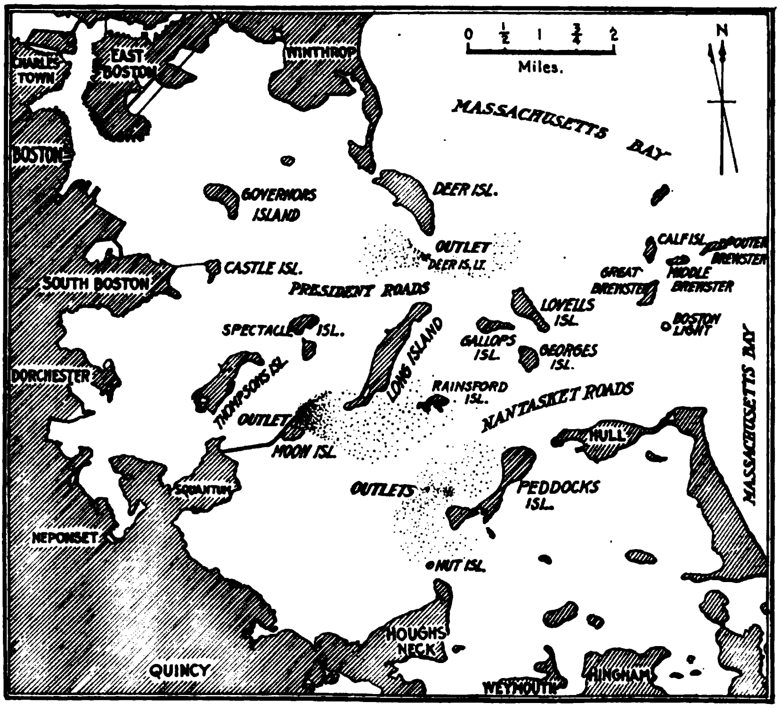


FIG. 35.—Outlets in Boston Harbor of main sewerage systems.

average velocity of 0.74 mile per hour, passing between Rainsford and Long Island, and at the turn of the tide reached a position between the Brewsters and George's Island and 4 miles from the point of starting.

Experience with this outlet shows that in general the floats showed the course which the sewage would take, but gave no indication of the great spreading tendency of the sewage in rising to the surface in salt water. That this was not fully appreciated at the time of the Moon Island experiments is indicated by the fact that rod floats 8 ft. long,

giving approximate mean instead of surface velocities, were used. In a report of the Massachusetts State Board of Health on "Discharge of Sewage into Boston Harbor" (1900), it is stated:

"It was decided to use much shorter floats than those used in previous investigations. The length of most of the floats was about 8 in. and the remainder 2 ft."

The rapid spreading effect is well shown by experiments in 1898 at Moon Island, recorded in a report of the Metropolitan Sewerage Commission on a high-level sewer for the Charles and Neponset Valleys. A float thrown overboard at the time the discharge started was 450 ft. behind after the edge of the sewage field had traveled 1800 ft., or was moving with a velocity only 75 per cent. of that of the edge of the sewage field.

The results of the discharge at this outlet are not entirely satisfactory, due largely to the storage and consequent stale condition of the sewage and the great area which it covers on being discharged intermittently from reservoirs instead of continuously. The following comments on the outlet were made by Clark in a report to the Metropolitan Sewerage Commission of Massachusetts:

"1. The area covered by a reservoir[1] of 22,000,000 gal. is approximately 750 acres.

"2. This area is enlarged according to the volume of sewage allowed to run from the outlet continuously, either before or after a reservoir discharge.

"3. A considerable time having elapsed after a reservoir discharge, a continuous discharge is held to a much narrower field than that occupied when 22,000,000 gal. are discharged within ¾ hour.

"4. When 11,000,000 gal. of sewage are allowed to run from the reservoir, and the gates are then closed, the area covered is not more than one-third what it is when 22,000,000 gal. are discharged at one time.

"5. The discoloration of the field of 750 acres at times of a full discharge is plainly marked on a comparatively calm day, and is objectionable to the sense of sight over about two-thirds of the area; but the offensive odors on a calm day are confined to a relatively small portion of the area.

"6. The presence of sewage is indicated at times by the presence of matters in suspension in the water for a distance of at least 1½ miles from the sewer outlet. Beyond this distance areas of sleek are sometimes visible, and occasionally areas containing distinct traces of sewage matter are seen.

"7. Observations of the rate of flow show that the sewage advances somewhat faster than the tidal flow for a few hundred feet from the outlet, owing to the impetus given by the high velocity in the outlet sewer.

"8. The observations and analyses show that, generally speaking, the upper 2 or 3 in. of the sewage area contain much the greater percentage of sewage for a considerable period, and that this percentage decreases from

[1] By reservoir is meant the contents of the reservoir on Moon Island in which sewage is stored for discharge on ebb tide.

the surface downward when the area is first covered. The percentage of sewage in the lower samples becomes less as the field moves forward and expands, and the sewage becomes more and more diluted on account of the greater area covered. The percentage of sewage in the surface samples also becomes less, although we are able to trace the presence of sewage at the surface longer than at the depth of a foot or more, on account of the large amount of sewage primarily present at the surface. It is also probable that there is a continuous sedimentation of the suspended matter in the sewage from the time the sewage is first discharged. This sedimentation takes place so gradually, and in a volume of water so large that samples collected at a depth of 5 ft. seldom show the presence of sewage, either by inspection or analysis.

"9. The temporary pollution of the sea water was mainly confined to the discolored area. This was usually found to have been broken up and dissipated in from 2 to 3 hours after the discharge of sewage, depending largely upon the force of the waves.

"10. The large areas sometimes noticed, because covered with a thin film of grease or so-called sleek, do not contain beneath this film enough sewage to be detected, and the film itself is hardly of appreciable thickness.

"11. The most noticeable odors blown toward the land from Moon Island appear to come from the sewage in the open reservoir, and not from the sewage after it has been discharged into the harbor." (Report of Metropolitan Sewerage Commission on High Level Sewer for the Charles and Neponset Valleys, 1899, page 92.)

In 1889, when an outlet for the North Metropolitan Sewerage District was sought, float experiments were made again, and a discharge point near Deer Island Light on the edge of the main ship channel was decided upon. From the experience gained at the Moon Island works, it was decided to discharge sewage continuously rather than only on the ebb tide. As a result, also, of studies at Moon Island, a very satisfactory prediction, as proved by results after the Deer Island works were constructed, was made as to the probable limits of the sewage field at different stages of the tide.

The Deer Island outfall discharges but a few feet below the surface, and as a result the surface water at times contains a considerable amount of sewage. In 1912 it served a population of about 558,000 persons. At times the large amount of sewage in the surface water near the outlet is disagreeable to persons in boats passing in the vicinity and to the keepers of the light nearby, and the State Board of Health has suggested that by an extension to deep water this condition could probably be remedied. The Metropolitan Sewerage Commission of Massachusetts received from Clark the following comment on the conditions at the outlet:

"1. At high water there is a sewage field lying south and west of the outlet. Immediately at the outlet, sewage can be traced to a depth of about 5 ft.

At the surface, in the densest part, there was about 30 per cent. of sewage. This percentage decreases rapidly as the distance from the outlet increases. At 900 ft. from the outlet only a small percentage of sewage was found, and at the edge of the visible sewage field the samples were nearly normal sea water.

"2. As the ebb-tide currents start, this field moves outward over Broad Sound, and is followed by a narrow band of sewage from the outlet. Samples taken directly over the outlet show that this bank has there a depth of about 5 ft., and that the densest part at the surface contains about 30 per cent. of sewage.

"3. Samples taken at intervals of 15 minutes show a steady decrease in the amount of sewage present, as follows:

Percentage of Sewage in Surface Samples: 15 minutes after leaving outfall, 20; 30 minutes after leaving outfall, 15; 45 minutes after leaving outfall, 5; 60 minutes after leaving outfall, 4.

"Less sewage is found below the surface, but distinct traces of it can be generally found at a depth of 2 ft.

"4. The discoloration has nearly disappeared when the sewage has moved 1⅛ miles from the outlet, which takes about 1¼ hours. Beyond this distance only the sleek can be found, and this only on calm days. The area covered by the discolored field is about 350 acres during the ebb-tide, but including the sleek the area is about 450 acres. Samples taken for analysis from the area of sleek show practically no organic matter, as the sleek itself is simply an exceedingly thin film of grease upon the surface of the water.

"5. A field of sewage collects on the west side of Deer Island spit at low tide. This field extends nearly to the Deer Island shore. The discoloration can be traced for about 1½ hours, reaching a distance of about 1 mile and covering an area of about 300 acres.

"6. The odors at the Deer Island outlet are much less noticeable than at Moon Island. This difference is due to the storage of sewage in the reservoir at Moon Island, as disagreeable odors increase when sewage is stored." (Report of Metropolitan Sewerage Commission on High-level Sewer, 1899, page 95.)

Still later a third outfall was constructed in Boston Harbor near Peddocks Island. It served a population of 382,000 in 1912. Experience at the Moon Island and Deer Island outlets led to the location of the new outlet in the bottom of the channel, instead of at a higher elevation, and to continuous rather than intermittent discharge of the sewage. The early experience with this outlet was stated as follows by the Massachusetts State Board of Health:

"This outlet is located at the bottom of the sea where the water is 30 ft. deep at low tide, and investigations during the past year indicate that the sewage is very quickly diluted by the sea water so that at the surface of the sea just over the outlet the percentage of sewage in the water as determined by chemical analysis is very small. This outlet, under ordinary conditions, would be very difficult to locate were it not for the fact that,

like the other outlets, it is a feeding ground for large numbers of gulls." (Report Mass. St. Bd. Health, 1910, page 18.)

FIG. 36.—Location of the outlet of the Salem sewerage system.

That a considerable amount of sewage from this outlet does come to the surface, however, is indicated by an experiment performed by the authors in 1911. Two pounds of aniline dye, eosin, was mixed with water and introduced into the sewer at the shore end of the outfall.

On proceeding to the vicinity of the outlet the colored sewage could be seen rising with considerable velocity and spreading out over an area of many acres. This experiment was performed during a very heavy wind and rain storm.

OTHER MASSACHUSETTS EXAMPLES OF DISCHARGE INTO SALT WATER

Salem.—Another shallow outlet which, it was predicted, from a study of currents, would be satisfactory, is at Salem (population in 1910, 44,000). Here the outlet is located near Great Haste Island near

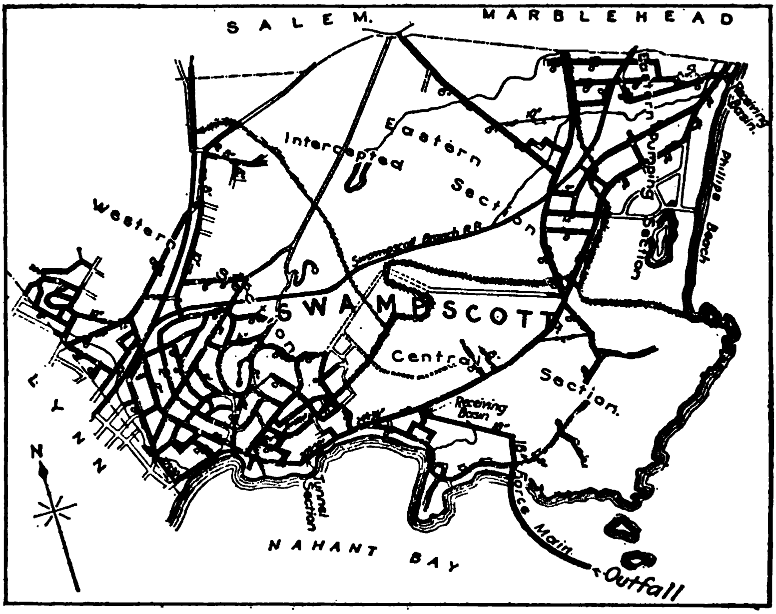


FIG. 37.—Location of the Swampscott sewer outlet.

the Main Channel, Fig. 36, and is reached by a submerged outfall sewer built by methods described in Volume I, page 324. Float studies by the Massachusetts State Board of Health led it to the conclusion that sewage could be safely discharged at this point at all stages of the tide, without danger that it would produce a nuisance on any shore. (1896 Report, page xlviii.) The outlet is near the surface at low tide, and the sewage spreads to long distances from it before becoming diluted thoroughly. The Board reported in 1910 that an offensive odor is sometimes noticeable over a wide area about the outlet.

The sewage contains much tannery waste and is very offensive. As a result, conditions are not satisfactory. The authors found the Beverly Harbor waters to be more contaminated by Salem than Beverly sewage.

Swampscott.—The most satisfactory sea outfall in Massachusetts is at Swampscott (population in 1910, 6200). Here the pipe is carried

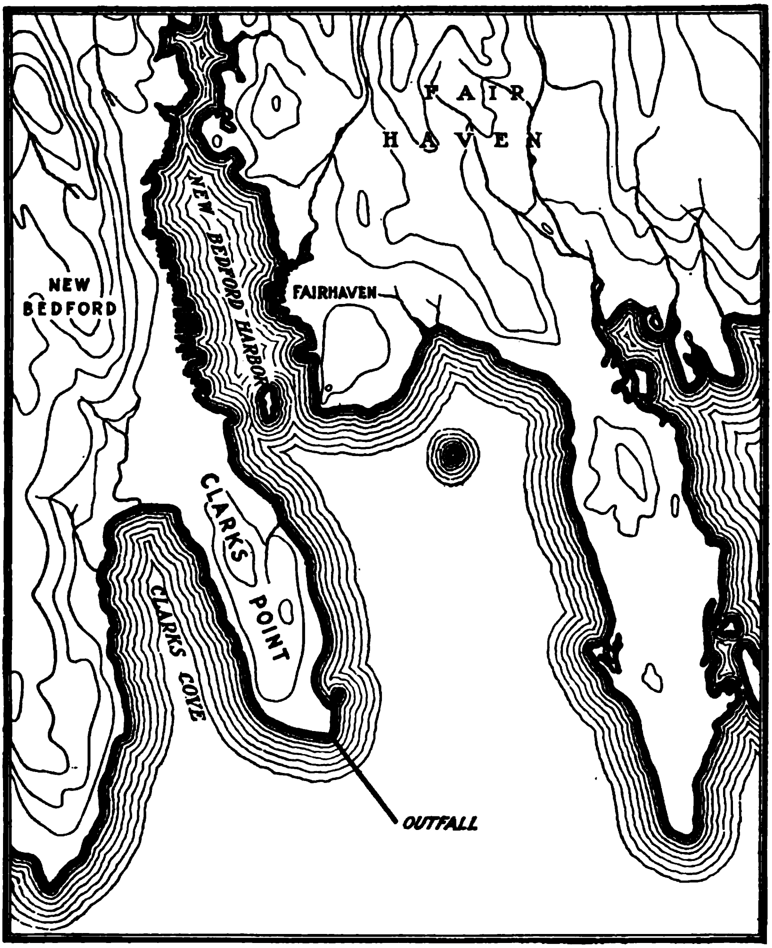


Fig. 38.—Location of the New Bedford sewer outlet.

several thousand feet from shore, Fig. 37, and discharges in 60 ft. of water, where the currents are from 3 to 4 miles per hour. According to the State Board of Health no evidence of sewage can be found,

even directly over the outlet. The quantity discharged is, however, not great and takes place practically in the open sea.

New Bedford.—The location of New Bedford (population in 1910, 97,000) is shown in Fig. 38. In 1910 the State Board of Health reported that about two-thirds of the city's sewage was discharged through numerous outlets into the harbor and the remainder through several outlets into the upper part of Clark's Cove. At the latter place the conditions were reported as very offensive and the nuisance was serious because the district was thickly populated. Many of the harbor outlets were also reported as very offensive. Preliminary plans were prepared by William F. Williams for discharging the sewage into tidal water at a depth of 30 ft. about 3000 ft. south of Clark's Point. The outlet was definitely located by observing the behavior of floats released at a point somewhat nearer the shore than that finally chosen. The object of these tests was to find a spot more influenced by the tide in the bay than that in the harbor. A few floats drifted close to the end of the point during flood tide and a southeast wind, and the State Board of Health accordingly advised moving the outlet about 1000 ft. eastward to its final location, with the upper surface of the outlet casting about 30 ft. below mean high water and 3300 ft. from the point.

OUTLETS IN RIVERS AND LAKES

Washington.—An outlet which was located only after careful study of currents and has given satisfaction in service, is at Washington, D. C. (population in 1910, 331,000). The sewage is discharged into the Potomac River at a point where it is subject to tidal influence, Fig. 39. In the study of the currents by Hering, Grey and Stearns, floats were considered of slight value, for observations could not be made at the lowest stages of the river, and they frequently stranded unless made so shallow as to be much effected by the wind. Reliance was placed on calculations based on the flow of the river and the tidal fluctuation, employing what is known as the "piston method" in which the total quantity of water moving up or down stream during each tide is figured.

The outlet was located at a point where it was considered that sewage would never be swept back to the city and would but seldom pass into the Eastern Branch or Anacostia River. It is about 700 ft. from shore in 25 ft. of water. The report of the Engineer Department of the District of Columbia for 1913 says that the outlet is in excellent condition at all stages of the tide; that the river bottom and beaches show no evidence of sludge or deposits; and that the surface is substantially free from oil and sleek at all times.

Auburn, N. Y.—A study of the effect of wind movement on the distribution of sewage through water was made in 1911 on Lake Owasco, from which the water supply of Auburn, N. Y. (population in 1910, 35,000) is obtained. This is a long, narrow lake having an average width of about 1 mile and a length of about 10 miles. Floats 5, 10, 15 and 20 ft. long, having their spindles reaching the surface and provided with large wings or sheets of metal which floated at the designated

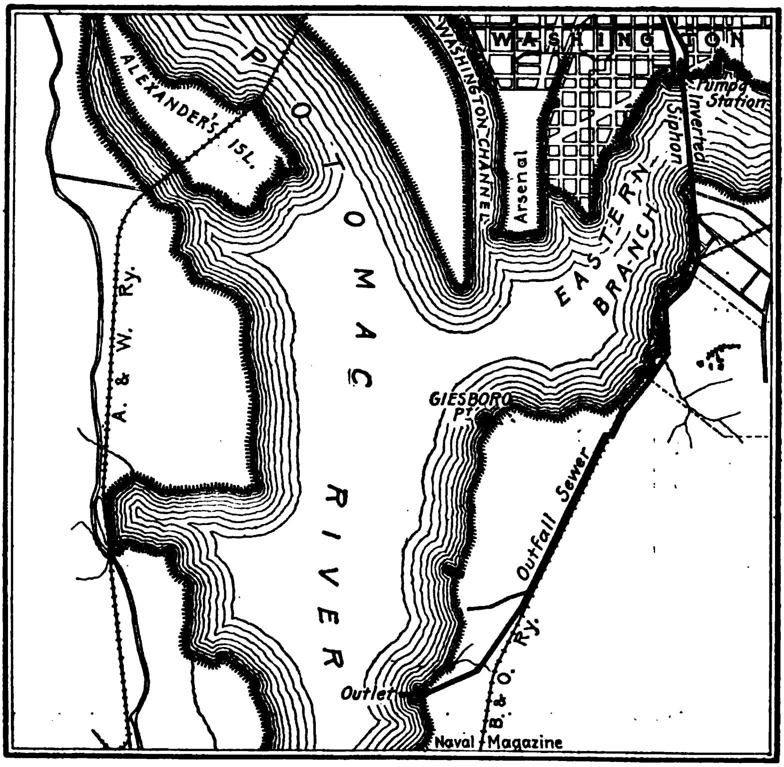


Fig. 39.—Location of the Washington sewer outlet.

depths, were used in the study. Their motion indicated a movement of the water at a depth of 5 ft. varying from 3¼ per cent. of the wind velocity when it was blowing 5 miles per hour to 1 per cent. when blowing 30 miles per hour. They also indicated that while at low velocities the currents at 5-ft. depths were two to three times greater than those at a depth of 20 ft., at high wind velocities the differences between the currents at these depths was not great. (*Engineering News*, August 16, 1913.)

Milwaukee.—The investigation in 1909–11 by Alvord, Whipple and Eddy of the methods of disposing of the sewage and protecting the water supply of Milwaukee (population in 1910, 374,000) included a study of the currents in Lake Michigan with the aid of the thermophone. As the capacity of the lake is about equal to its discharge during 100 years, the lacustrine current toward the outlet at the Straits of Mackinac is inappreciable, and temperature and winds are responsible for both surface and lower movements of the water as explained on page 261.

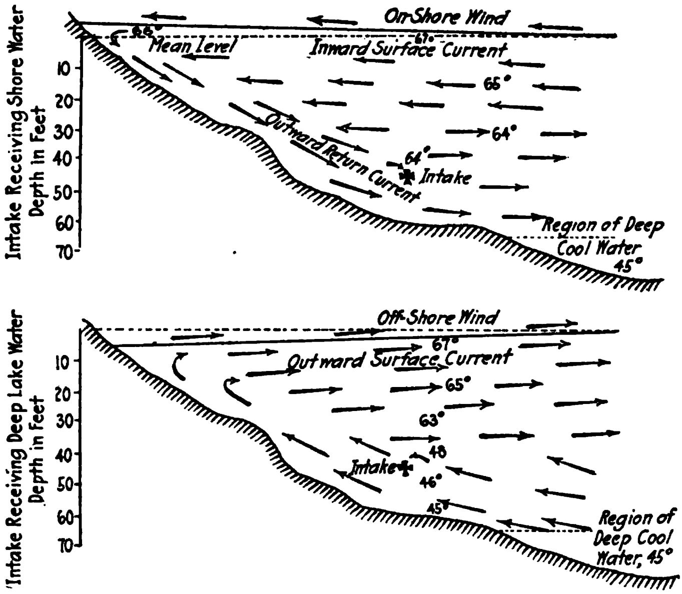


FIG. 40.—Effect of wind on lake currents at Milwaukee.

There is a general local opinion that the surface water near the head of Lake Michigan has a general drift southward along the west shore and northward along the east shore. The Weather Bureau records show that the resultant direction of the winds in that region would have such an effect.

A much more important effect of the winds is shown in Fig. 40, illustrating the reason for the sudden large fluctuations in the temperature of the water drawn in at the intake of the Milwaukee water works. Thermophone readings of the water every 5 ft. in depth at the intake and at points 1, 2 and 3 miles east of it showed that 2 or 3 miles from shore, where the lake becomes deep, the water below a depth of about 65 ft. is thermally stratified during the summer and practically

stagnant. Nearer the surface, however, the wind has a marked effect in circulating the water in the manner shown.

The temperature observations were supplemented by investigations of the bacteria in samples of water collected at numerous points, both at the surface and at the bottom. The results are given in Figs. 41,

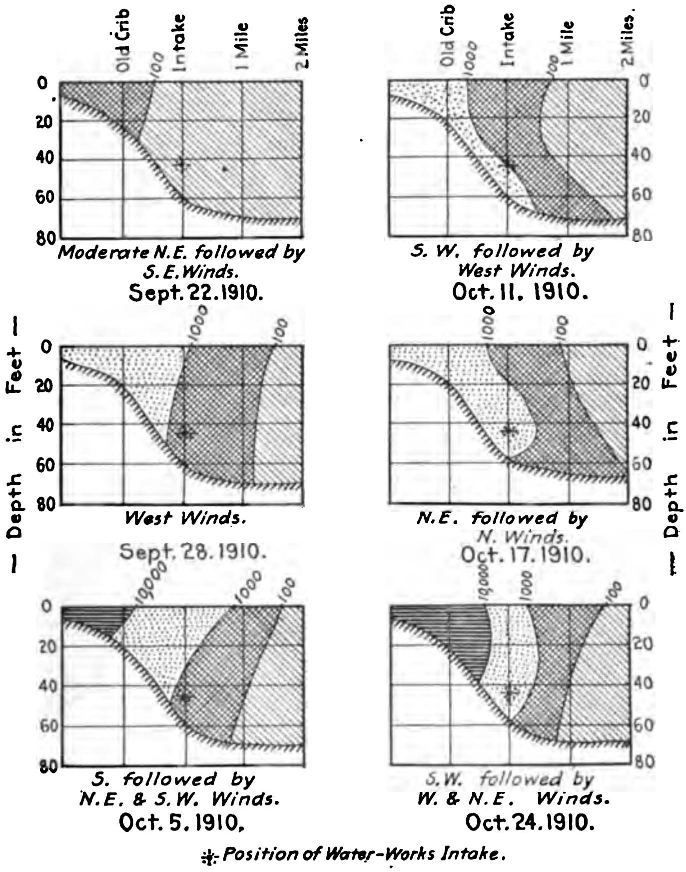


Fig. 41.—Effect of winds on vertical distribution of bacteria in a plane passing east and west through the intake of the Milwaukee water-works.

42 and 43, and the effect of the polluted water of the river on the lake water into which it was discharged is clearly shown.

On September 22, 1910, there was an on-shore wind. The bottom water had practically the same temperature as that at the surface, and the turbidity of the water was the lowest observed at any time during the investigation. The surface water containing the largest amount of

chlorine was driven to the northward and kept near the shore, and the largest number of bacteria were found near the shore.

On September 28, there was a light off-shore wind, and there was little difference between the surface and bottom temperatures. The turbidity was high on this day. The chlorine extended more directly from the mouth of the river, whence the pollution came, and was more

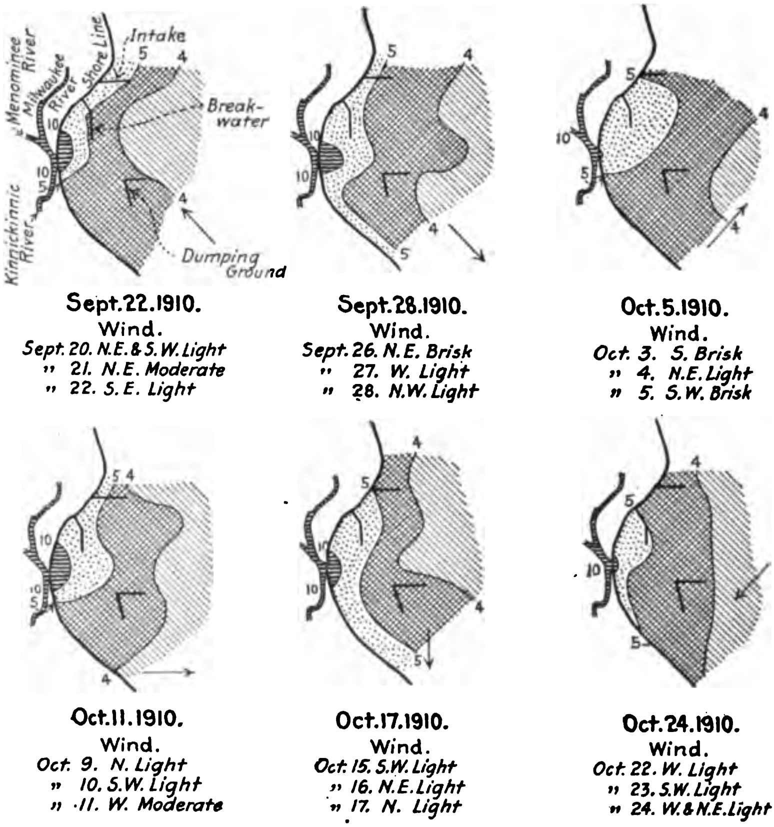


FIG. 42.—Effect of winds on distribution of chlorine in surface waters of Milwaukee Bay.

widely distributed in the bay. The surface water containing the largest number of bacteria was now spread out from the shore.

On October 5, the wind was off-shore and colder water was brought in at the bottom, the difference between the average surface temperature and the lowest bottom temperature being 13.2°F. The surface water containing the largest amount of chlorine had been driven

to the north and there was a well-marked surface drift of water with large numbers of bacteria toward the water works intake.

On October 11, with off-shore winds, the temperature conditions, chlorine and bacteria were about the same as on October 5.

On October 17, with a north wind, the difference between the average surface temperature and the minimum bottom temperature was 3.3°.

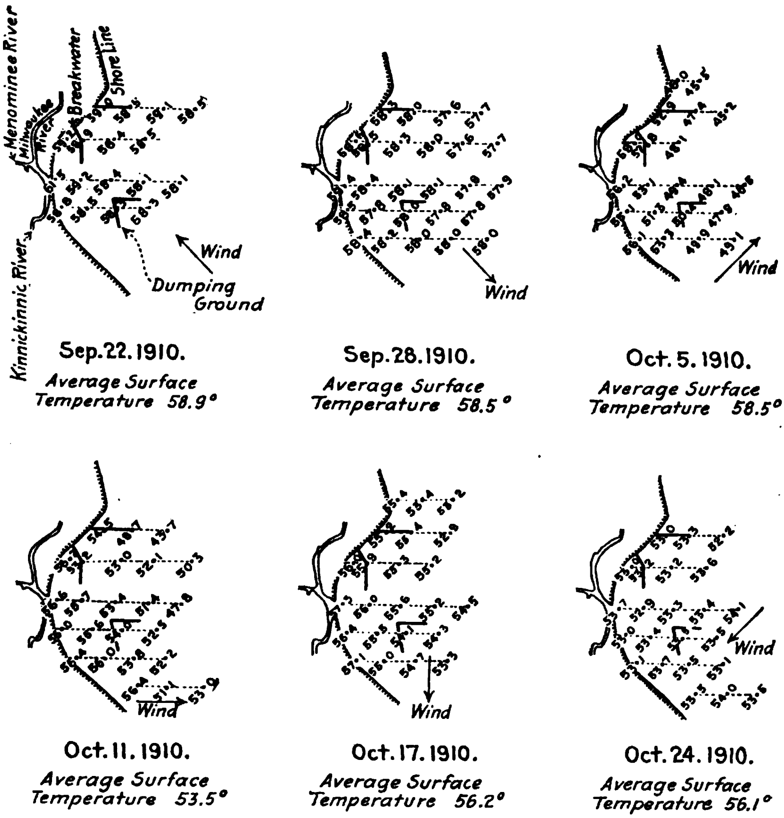


FIG. 43.—Effect of winds on bottom waters in Milwaukee Bay.

The surface water with the largest amount of chlorine and numbers of bacteria had been driven southward.

On October 24, there was an on-shore wind and the difference between the average surface temperature and that of the bottom water was only 1.3°. The turbidity was high, the largest proportion of chlorine was near the shore, but the distribution of bacteria was unusual and possibly due to heavy pollution caused by first flushings by about ½ in. of rain on October 21 after a long period of dry weather.

There seemed to be a general relation, not close, between the turbidity of the water and the number of bacteria. From the mouth of the river to a point somewhat beyond the breakwater, the water had a distinct sewage odor, then the odor became faint and finally it was the very faint vegetable odor of the normal lake water. Moldy odors were occasionally discovered in the bay. All the odors were platted and the charts confirmed the local opinion that there was a southerly drift of water at this place. The normal chlorine of the lake water is about 4 parts per 1,000,000; at the mouth of the river it averages 11.8 parts at the surface and 10.3 parts at the bottom, and there is a district of heavy pollution near the river mouth at all times, as shown in Fig. 42. Within the large zone where the chlorine is between 4 and 5 parts per 1,000,000, the pollution varies, and as the water works intake lies within it, the effect of the winds upon the degree of bacterial pollution in this district is particularly important. It was found that the last large wind movement had more influence on observed conditions than a light wind at the time of sampling. It was also discovered that after on-shore winds there were four times as many B. coli in the bottom water as in the surface water, but with off-shore winds the surface water contained eight times as many bacteria as the bottom water.

As a result of their investigations, Alvord, Whipple and Eddy recommended the immediate filtration of the local water supply, with disinfection when necessary, and sewage treatment carried only far enough to prevent undue contamination of the rivers and bay.

Cleveland.—In 1904, an investigation of the pollution of Lake Erie near the intakes of the water works of Cleveland (population in 1910, 561,000) was made by Whipple. The general current in the lake was found to be about a mile in 6 days, which is negligible in comparison with currents due to the winds. During 11 months of the year the prevailing winds are down the lake; the winds across the lake are practically equal in both directions. The strongly polluted water of the Cuyahoga River (Volume I, Fig. 2), was found to pollute the lake more than the outlets along the lake front, and the greatest danger of an infection of the water supply Whipple held to be the possible direct flow of the polluted river water across the harbor to the intakes.

The investigations showed that the turbidity of the water did not have an appreciable relation to the amount of sewage pollution, although the numbers of bacteria varied with the turbidity, decreasing rapidly off-shore. The chlorine determinations indicated a general outward sweep of the polluted water away from the shore toward the eastward. Wind was found to have the same effects that have been described in connection with the Milwaukee investigation. The odor of the lake water near the shore was frequently moldy and oily, but 2 or 3 miles off-shore little odor was perceptible. The results of the

investigation, Whipple stated, pointed to a general dispersal of the sewage through the bay, so that at times slight traces of it reached the new intake. The water at the old intake was declared to be infected with pathogenic bacteria. The information which was obtained led Whipple to conclude that for some years after the completion of the proposed intercepting sewer, the lake water at the new intake would be rarely, and only in a very slight degree, influenced by the pollution of the city. This danger will be reduced by the construction of the sewage treatment works now planned. (Report on Cleveland Water Supply, 1905.)

Rochester.—The effect of discharging the sewage of Rochester (population in 1910, 218,000) into Lake Ontario was investigated by Whipple in 1912. The dry-weather sewage at that time was about 20,000,000 gal. per day. It was discharged into the Genesee River about

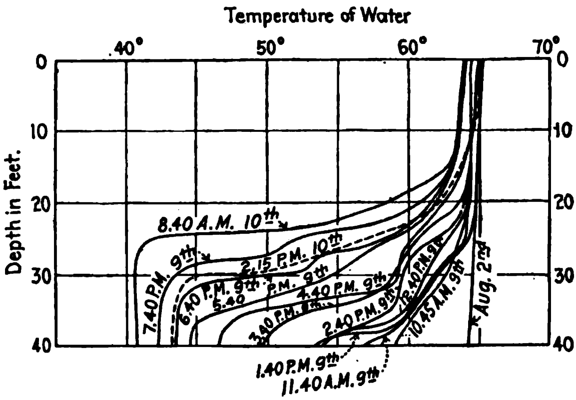


Fig. 44.—Changes in temperature of Lake Ontario water 6000 ft. from shore, August 2–10, 1912.

7 miles above its mouth, where the dilution during ordinary low river stages was only 1.4 cu. ft. per second per 1000 persons, and offensive conditions resulted.

The observations as a whole showed a very small number of bacteria in the lake water more than ½ mile from the river mouth, which was ascribed to the death of most of the sewage bacteria during their passage down the river. The distribution of the survivors in the lake was found to be controlled by the wind, and varied greatly from hour to hour. As at Milwaukee, the bacterial contamination along the shore was high at times, indicating some danger in bathing in the water. Where there were less than 100 bacteria per cubic centimeter, the water was usually odorless, but when the number was over 100 a musty or moldy odor was perceptible.

Thermophone observations of the temperature at different places and depths gave the same results as similar tests at Milwaukee. Results obtained on different days at a point 6000 ft. from shore, where the water was 40 ft. deep, are shown in Fig. 44. Northerly winds prevailed before August 2, and the bottom currents were outward. Between August 7 and 11 strong southerly winds changed the condition. On August 9 cold water was moving rapidly toward shore. Between 10.45 a.m. and 7.40 p.m., the bottom temperature fell from 59° to 43° and on the following day to 41°. On August 10 a slight change in the wind to the north caused the temperature to rise to 43° in a few hours, and a few days later, after the wind had been blowing from the north, the temperature of the bottom water had risen to 65°.

The outlet of the new outfall sewer is located in the lake 1000 ft. beyond the point where the results shown in Fig. 44 were obtained. The sewage will be screened and settled before its discharge. Whipple's application of the results of the investigation to the proposed outlet was as follows:

"If the warm summer sewage were discharged into the cold bottom water, as it would be with an off-shore wind, being lighter, it would rise quickly toward the surface and be caught by the outward surface current and carried away from the shore. Only a comparatively small proportion of the sewage would reach the shore under these conditions. Just what proportion would be carried toward the shore cannot be calculated from the data at hand, but it seems certain that the amount would be very small. Thus the effect of southerly winds which ordinarily prevail at this season of the year would be to carry the sewage out into the lake.

"With the warm surface water flowing out at the bottom, the sewage would enter bottom water of approximately its own temperature; at times the sewage might be even colder than the water. Under these conditions there would be less tendency for the sewage to rise to the surface, and a large proportion of it would be carried outward by the outgoing bottom current. Even if the temperature were slightly warmer than the bottom water of the lake at the outlet, the difference in specific gravity would be so small that sewage would rise slowly into the upper currents, so that the amount carried toward the shore under these conditions would be relatively small, although probably greater than during the prevalence of off-shore winds.

"The time when the sewage would be most likely to be carried toward the shore during the summer would be when a period of calm was followed by northerly winds. This would cause first a concentration of sewage at the surface near the outlet, which would gradually drift inshore. Under these conditions, however, the diffusion would be great.

"During cold weather different conditions from the above would obtain. The surface water of the lake would have a temperature not much above 32°, but the bottom water would still maintain its temperature of maximum density, *i.e.*, 39.2°, while the temperature of the sewage might be between these two figures. With on-shore winds the bottom water at the sewer

outlet would be colder than the sewage, and being below maximum density, would be lighter. Therefore the sewage would tend to remain near the bottom and flow out as a bottom current. With off-shore winds the water at the sewer outlet would be warmer and heavier than the sewage, so that the latter would rise toward the surface and be carried outward by the surface currents.

"At intermediate seasons of the year the sewage would doubtless be warmer than the water into which it was discharged, so that it would tend to rise and its distribution would then depend more upon the direction of the surface currents than upon the direction of the currents at the bottom. Under certain conditions, therefore, the on-shore wind might tend to carry the sewage toward the shore. These conditions, however, would not be likely to obtain during the season when the beaches are visited by pleasure seekers and the lake water used for bathing." (Report on Sewage Disposal System of Rochester, 1913, page 191.)

The location of the outlet was approved after critical study by the State Commissioner of Health.

CHAPTER VIII

GRIT CHAMBERS

The solids in suspension in sewage have been discussed in Chapter V and the general purpose of grit chambers was explained in Chapter VI. The extent to which storms affect the quantity of solids in sewage which can be removed by grit chambers was indicated by testimony before the Royal Commission on Sewage Disposal. (Appendix I, Fifth Report.) During a 4-hour storm at Burnley, for example, Pickles and Ross reported that at the ends of the first, second, third and fourth hours of the rainfall the suspended matter amounted to 248, 800, 256 and 56 parts per 1,000,000. At Heywood, the suspended matter during the first rush of a protracted storm was reported by Joshua Bolton to be 2380 parts per 1,000,000, and at the ends of successive hours it was: first hour, 1110; second hour, 690; third hour, 500; fourth hour, 380; fifth hour, 330; sixth hour, 280; seventh hour, 180.

There are parts of this suspended matter, in cities with combined systems of sewers, which are so coarse and heavy that they settle quickly and can be removed by grit chambers. They are of mineral nature and, except for the organic matter that may collect on their surface, they are unlikely to become offensive when removed from the sewage and exposed to the air. If too long a period of subsidence or too low a rate of flow of sewage is permitted, particles of organic matter will settle with the grit and render it offensive when exposed to the air. In addition to this relatively coarse mineral matter, the suspended matter also includes coarse organic matter which can be removed, if desired, by screening devices described in the next chapter.

There are several conditions which may make it desirable to remove the coarse mineral matter, such as: (1) to prevent injury to pumping machinery by grit, (2) to keep mineral matter from the sludge of settling basins, which flows less readily if coarse grit enters it, (3) to prevent inverted siphons from becoming clogged with material difficult to remove in many cases, (4) to prevent deposits of silt in receivers of sewage, an important purpose along some English rivers,[1] (5) to keep

[1] "One point in regard to which we think that strict regulations will usually be found necessary is the reduction of the suspended solids of an effluent to a low figure, before that effluent is allowed to escape into a water-course. As is well known, these suspended solids are always—or practically always—putrescible. . . . As a general rule, we consider that the harm due to suspended solids would not be serious, if the effluents contained not more than 3 parts of suspended solids per 100,000, and it is shown by experience that this would

heavy grit from settling on the sludge in septic tanks and sealing it so that when ebullitions of the gases of decomposition occur they are very violent.

Quantity of Grit.—The quantity of grit collected in a grit chamber will depend on the character of the sewage, the dimensions of the chamber, the specific gravity of the grit and the rate of flow of the sewage. These factors vary widely, and consequently the reports of the quantities of grit removed from the sewages of different cities by these chambers rarely show much agreement.

Formerly there was little information to guide the engineer in selecting the dimensions for his chambers, but experience now indicates that by checking the velocity of flow to about 1 ft. per second little organic matter will settle with the grit. This is the velocity adopted in some designs in this country and used in designing chambers of plants in the Emscher District in Germany. Dr. Karl Imhoff informed the authors that the grit intercepted at these plants in 1913 averaged 6.5 cu. yd. per 1000 persons. Caution should be exercised in the use of data regarding the volume of grit, for while the evidence varies greatly there are often local conditions which will mislead if not given due weight. For example the amount removed at velocities of only 0.2 to 0.6 ft. per second, averaging 0.5 ft., from the grit chamber at Worcester, Mass., was only 3.2 cu. yd. per 1000 persons in 1913, about half that collected in the Emscher district, where the velocity was twice as great. At Worcester, however, most of the flow during heavy storms passes to the rivers through storm overflows and in 1913 only about 31 per cent. of the sewer system was of combined sewers. Storm drains discharge their entire flow directly into the rivers.

The Worcester grit chamber was designed in 1904. When the chemical precipitation plant there was built, no provision was made for intercepting the sand and gravel carried by the sewage, particularly during storms. Accordingly these substances were carried into the precipitation basins and settled with the sludge. When the sludge was removed it was very difficult to force the heavy grit to move with it, and this grit was therefore allowed to accumulate in the basins, from which it was removed annually at a considerable expense for labor. Since the chamber was put into service, the average amount of grit intercepted by it annually, expressed in cubic yards per million gallons of sewage, has been as follows:

Year..	1905	1906	1907	1908	1909	1910	1911	1912	1913	Ave.
Grit...	0.20	0.12	0.11	0.15	0.13	0.11	0.12	0.10	0.09	0.125

not be an unreasonable figure to demand in the majority of cases. There are, of course, cases where a larger quantity of solids might be safely allowed in an effluent, while in some others even 3 parts might be too much." (Fifth Rep., Royal Sewage Com., 1908, page 219.)

The basins of the chamber are allowed to become nearly filled with deposit before cleaning, and are cleaned in rotation once in 4 to 8 weeks. The grit is extremely offensive and requires immediate covering to prevent escape of the odors.

European information concerning the quantity of material properly intercepted from sewage[1] in grit chambers was summed up by Frühling in his "Entwässerung der Städte" in 1910 as follows:

"The amount of sediment which can be intercepted in grit chambers varies between quite widely separated limits, according to local conditions. For example, in 1904, 10,130 cu. yd. of sand, 5.04 cu. yd. per 1000 persons per year, were taken from the grit chambers of the sewage pumps in Berlin. The figures for the different sewerage districts, which have populations equivalent to those of cities of medium size, show fivefold variations among themselves. The largest amount, 11.9 cu. yd., was obtained in the section with heavy travel and many asphalt streets which require frequent sanding. Frankfort approached it, with an average annual deposit of 2929.8 cu. yd. in the grit chambers in 1897–1903, equivalent to 10.9 cu. yd. per 1000 persons per year. If Paris, in spite of the absence of catch-basins and with many stone-paved streets, fell behind the above figures with its 6.5 cu. yd., it is to be remembered that not less than 11.8 cu. yd. of grit and sludge per 1000 persons were taken annually from the sewers themselves. The more care paid to intercepting the sediment, the better the street cleaning, the more favorable the grades of the sewers and the more thorough the flushing of the sewers, the smaller will be the need of cleaning sewers. In Berlin, where very flat grades are employed, only the smaller part of the sediment entering the sewers formerly reached the grit chambers of the pumping stations. In 1904 the percentage had risen to 53.3 while only 46.7 per cent. or 8881 cu. yd. was removed from the sewers. With improvements in the flushing the 5.04 cu. yd. mentioned above will doubtless be raised, although it is hardly probable that the limit of $5.04/0.533 = 9.4$ cu. yd. will be reached.

"In Wiesbaden the grit chambers removed 1072 cu. yd. in the year beginning April 1, 1901, or 11.2 cu. yd. per 1000 inhabitants annually, while in Cologne a daily average of 3.3 cu. yd. per 1000 persons was intercepted annually. This latter amount is very small in comparison with the figures previously reported; this is due to the freedom from grit of the Cologne sewage, largely because a part of the grit is intercepted by gutter-shaped grit pits, about 3.28 ft. deep and 9.02 ft. long, in the sewer inverts, which are spaced about 0.62 mile apart and are cleaned every 2 to 4 weeks. The stretches of sewer between the pits remain clean.

"If it is attempted to draw an average from these and similar figures, it may be assumed that a well-planned and managed sewerage system will yield about 6.5 cu. yd. grit per 1000 persons annually."

[1] In this connection the greater strength or concentration of the German sewage as compared with American sewage is to be clearly borne in mind, and also the relative use and care of catch-basins in Germany and the United States.

The amount of grit removed by the grit chamber on the north bank of the River Elbe at Hamburg is about 0.17 cu. yd. per 1,000,000 gal. of sewage or 3.5 cu. yd. per 1000 population annually. ("Kanalisation der Stadt Hamburg," Merckel, page 216.) The grit and screenings are removed together on scows from the Hamburg plants to an island 19 miles down the river, where they are used as a fertilizer by farmers. The city pays them 7 cents per cubic yard for taking the material, according to Kenneth Allen.

At Manchester, England, from 0.3 to 0.45 cu. yd. of grit per 1,000,000 gal. of sewage are removed annually, according to Emil Kuichling. The amount removed from the "deposit sewers" of the Boston main drainage system, described later in this chapter, was 0.25 cu. yd. per 1,000,000 gal. in 1909 and 0.32 cu. yd. in 1904.

TABLE 59.—SUSPENDED MATTER REMOVED FROM SEWAGE AT FRANKFORT, GERMANY, BY GRIT CHAMBERS AND SCREENS

(Uhlfelder and Tillmans, *Mit. König. Prüf. Ans. f. Was. u. Abwasser.*, 1908)

Kind of sewage Hours covered	Day 8 a.m.–2 a.m.	Night 2 a.m.–8 a.m.	Average 24 hours
RAW SEWAGE (parts per million)			
Dissolved solids, total	783	615	741
Organic	230	141	208
Mineral	553	474	533
Suspended solids:			
Total	485	191	411
Organic	285	110	241
Mineral	200	81	170
Total solids:			
Total	1268	806	1152
Organic	515	251	449
Mineral	753	555	703
GRIT REMOVED (parts per million)			
Total	65	61	64
Organic	39	32	37
Mineral	26	29	27
SCREENINGS (parts per million)			
Total	28	5	22
Organic	24	5	19
Mineral	4	0	3
GRIT AND SCREENINGS (parts per million)			
Total	93	66	86
Organic	63	37	56
Mineral	30	29	30

In the course of extended investigations at Frankfort, a number of determinations were made of the removal of suspended matter by the grit chambers and screens at that place, which have been summarized by Allen as follows:

"Measurements made about 7 years ago indicate that 16 per cent. of the suspended matter is removed in the grit chamber and 10 per cent. more by the screens, making 26 per cent. in all. At that time the water supply was 47 gal. and the sewage flow 39½ gal. per capita. Table 59 shows the quantities of material removed by the grit chamber and the screens, as given by Uhlfelder and Tillmans. Of the whole quantity, about three-quarters were removed by the grit chamber and one-quarter by the screens. The grit was about half organic and half mineral, and the screenings were from 86 to 100 per cent. organic. These relations, of course, depend on the design and operation of both grit chambers and screens; but, at Frankfort, although the grit chamber removes three times as much material as the screens, the latter remove half the organic matter taken out." (*Proc. Am. Soc. C. E.*, August, 1914, page 1862.)

Character of Grit.—The offensive grit taken from the chambers at Worcester was reported by H. P. Eddy in 1906 to weigh from 1848 to 2430 lb. per cubic yard, averaging 2014 lb. The dry solids ran from 53.3 to 75.4 per cent., averaging 64.1; the loss on ignition from 18.9 to 28.1 per cent., averaging 22.7; the organic nitrogen in dried samples from 0.515 to 0.871 per cent., averaging 0.675.

The grit and screenings removed at the works on the north bank of the Elbe at Hamburg have the composition given in Table 60. The average material from German grit chambers, according to Frühling,

TABLE 60.—PERCENTAGE COMPOSITION OF GRIT AND SCREENINGS, NORTH BANK WORKS, HAMBURG

(Merckel, "Kanalisation der Freien und Hansestadt Hamburg," page 216)

	Grit			Screenings	
Water	65.35	92.62	85.24	22.87	55.90
Nitrogen	0.52	0.27	0.02	0.21	0.27
Phosphoric acid, P_2O_5	0.52	0.18	0.12	0.40	0.29
Fats, ether extracted	1.83	1.68	5.11	0.44	1.28
Ash	20.28[1]	1.10[2]	5.16[3]	71.28[4]	37.15[5]

[1] Mostly sand. [2] Sand, 0.32. [3] Sand, 2.88. [4] Sand, 67.86. [5] Sand, 35.33.

consists of about 33 per cent. of water, 3 per cent. of organic matter, and 64 per cent. of mineral matter. ("Entwässerung der Städte," page 466.)

The grit removed at the Davyhulme works of the Manchester, England, system was stated in the report for 1902 to consist of a "large

proportion" of sand, fine coal and cinder, a "fair amount" of coal in moderate-size pieces, also gravel, clinker, small pieces of brick, wood and leaves. Altogether there was about 35 per cent. of combustible matter in the dried samples, of which about 7 per cent. was coal which could be easily picked out by hand.

About 4800 lb. of wet grit removed from Boston sewage in the spring of 1904 by Winslow and Phelps was reported by them to contain 1300 lb. of water, 570 lb. of clean stone and 2900 lb. of fine dry detritus which showed a loss of 319 lb. on ignition, 6.6 lb. of organic nitrogen and 5.1 lb. of oxygen consumed. Expressed in parts per million of sewage, these figures become, wet detritus, 190; water, 52; clean stone, 23; total fine dry detritus, 117; loss on ignition, detritus, 13; organic nitrogen, detritus, 0.26; oxygen consumed, detritus, 0.2.

As a rule the information regarding suspended matter obtained at sewage testing stations is very misleading owing to the failure of the samples to contain an average amount of such matter. The heavy mineral suspended matter is probably not uniformly distributed through the sewage but is more concentrated toward the bottom.

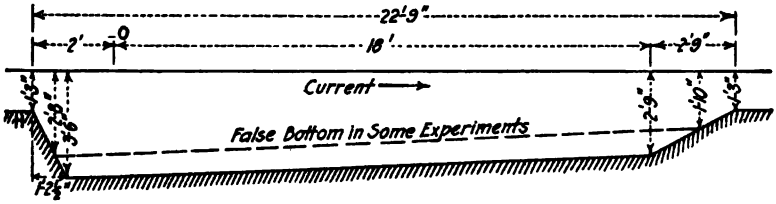


FIG. 45.—Experimental grit chamber, Rochester, N. Y.

Effect of Velocity of Flow.—During 1909, numerous experiments to determine the best form of sedimentation basins were made by N. A. Brown under the direction of Emil Kuichling, consulting engineer, and E. A. Fisher, then city engineer of Rochester, N. Y. A channel of sheet steel about 64 ft. long and 10 in. wide was used, in which water could be circulated at different velocities. At one point in this channel a grit chamber, Fig. 45, was constructed. Clean water was circulated through the channel and measured by discharging it into a large can on a platform scale, so that the discharge was gaged by both volume and weight. Surface velocities were determined by a paraffin ball 1 in. in diameter. Velocities below the surface were determined by a submerged float, the lower part consisting of a cross of two hard rubber plates, each $4\frac{3}{32} \times 1\frac{23}{32} \times \frac{1}{16}$ in., suspended by a waxed silk thread from a vertical frame about 2 ft. high, which could be moved along the tank. This float was raised or lowered until a depth was found at which no current was indicated. The measurements were made at every foot of the length of the tank to the right of the point *O*; to the left of that

point there was a strong eddy which caused the submerged float to rise into the current moving downstream. Sand was prepared by washing and screening it, and was placed on the bottom of the channel a foot or so above the head of the tank. In some of the experiments a false bottom was placed in the tank, as shown in the illustration.

The position of the plane of no velocity in the tank varied little for different velocities in the channel; in all cases it extended from 16 to 19 in. below the surface at *O* and reached the bottom of the tank about 13 ft. from *H*. With the false bottom and a lower water surface, it was parallel with the plane just mentioned and reached the bottom at distances of 6 to 8 ft. from *H*. The experiments are recorded in detail in Fisher's "Report on the Sewage Disposal of Rochester," 1913, and the more important results are summarized in Table 61. It is evident that they are rather unreliable.

TABLE 61.—EXPERIMENTS AT ROCHESTER TO DETERMINE AMOUNT OF GRIT CAUGHT IN TANK

Size of sand, inches[1]	Deep tank			Tank with false bottom		
	Velocities, ft. per sec.		Grit caught in tank, per cent.	Velocities, ft. per sec.		Grit caught in tank, per cent.
	In channel	In deepest part of tank		In channel	In deepest part of tank	
0.0076–0.0107	0.75	0.19	80	0.85	0.19	92
0.0076–0.0107	0.86	0.22	98	1.08	0.24	86
0.0076–0.0107	1.01	0.25	71	....	....	....
0.0076–0.0107	1.02	0.26	87	....	....	....
0.0055–0.00675	0.36	0.09	83	1.18	0.28	88
0.0055–0.00675	0.69	0.25	91	1.29	0.29	88
0.0055–0.00675	0.98	0.25	92	1.37	0.31	84
0.0055–0.00675	1.16	0.29	94	....	....	....
0.003–0.0055	0.31	0.08	77	0.86	0.19	90
0.003–0.0055	0.77	0.19	86	1.04	0.24	88
0.003–0.0055	0.82	0.20	90	....	....	....
0.003–0.0055	0.97	0.24	78	....	....	....

[1] In millimeters these sizes are: 0.193–0.272; 0.140–0.171; 0.076–0.140.

DETAILS OF DESIGN

Velocity of Flow.—The velocity of flow of the sewage through the grit chamber is an important factor to be considered in its design. A discussion of the carrying power of flowing water is given on page 108

of Volume I of this treatise. The Rochester experiments with a model grit chamber confirm the general deduction from available experimental information and service experience, that a mean velocity of approximately 1 ft. per second is about right, unless it is necessary to remove very fine and light material, in which case it may be desirable to reduce the velocity and run the risk of some offensive organic matter settling with the grit. The German views are summarized by Frühling as follows:

"The rate of flow through the grit chamber should be so low that the grit carried by the sewage into it will be intercepted. A guide in fixing it is afforded by the consideration that the movement of coarse sandy particles on the smooth invert of a sewer begins when the mean rate of flow of the sewage is 2 ft. per second, while the smaller particles require a lower rate; the limit for fine sand is apparently about 10 in. In order to hold back all sand, the rate of flow must fall below these limits, but this is only approximately practicable to attain because of the variable quantity of sewage. Moreover, it must not be overlooked that the lowering of the rate of flow promotes the subsidence of organic suspended matter and a corresponding tendency toward the collection of an offensive sediment. Therefore it is generally desirable to employ a somewhat higher rate, if the object is not solely to prepare the sewage for pumping." ("Entwässerung der Städte," 1910.)

The range of velocities in feet per second in five German grit chambers is given in A. Elliott Kimberly's translation of Schmeitzner's "Clarification of Sewage" (page 51) as follows: Aachen, 0.06; Elberfeld, 0.17; new Hamburg works, 1.0; Wiesbaden, 1.64; old Hamburg works, 3.28. These figures show clearly how little guidance in design is to be obtained from any but recent installations in that country. Dr. W. P. Dunbar's statement in his "Sewage Treatment" may prove dangerous advice to follow unless the engineer is willing to have organic matter mixed with the grit; it reads:

"The dimensions of detritus tanks are generally so arranged that the velocity of the sewage as it passes through them is not more than 2 in. per second. As the tanks are usually very short, the sewage only remains in them for a few minutes" (page 46).

The fluctuations in the volume of sewage are so large that it is generally considered desirable to provide at least two independent basins in a grit chamber. One of these will be sufficient for the dry-weather flow and moderate rainfalls, affording while in use an opportunity for cleaning the other, and when very large volumes of sewage reach the chamber both basins may be used simultaneously. In Fig. 50 three parallel basins are shown, but these are exceptionally narrow and shallow.

There has been a tendency to leave out of consideration in estimating

velocities, the cross-section of the pit in which the grit is collected. It is doubtful if this does not sometimes lead to error, even though it is clear from the Rochester experiments and earlier German investigations that there is a certain plane within the pit on which there is practically no motion and below this plane the motion may be in a direction contrary to that of the main stream of sewage in the top of the basin. The shape of the chamber, particularly the length and depth of the pit, influences the extent of the total cross-section in which there is an onward motion of the sewage, but there is practically no definite experimental information to guide the designer in determining what area to use. The subject is important, because, unless care is taken in estimating the velocity, particularly at times of small discharge, the actual rate may fall considerably below that assumed and an undesirably large amount of organic matter may mingle with the grit. A cross-section with vertical walls and V-shaped bottom is a good one for this class of structures on account of its hydraulic properties, which are indicated by Fig. 144, Volume I, page 399.

Cross-section.—In addition to giving the grit chamber a cross-section which will result in proper velocities, the designer must adopt a section which can be easily cleaned and, where mechanical removal of the grit is employed, will cause the grit to collect in valleys or hoppers. Cleaning is facilitated by rounding the intersections of the walls and bottoms and avoiding all angles in which sludge can collect. Where the grit is to be removed by shovel, the bottom of the pit receiving it may be horizontal, but if mechanical cleaning will be followed, the bottom should slope to a valley as in Fig. 48, or a hopper. At most places where such construction has been adopted the bottom slopes are planes, but at Prague they are curved. In some of the early trough and hopper bottoms, the slopes were too flat for the grit to slide down them, and there is some evidence to the effect that the inclination should be at least 45 deg.

In a large plant where the cost of construction may be influenced by difficult foundation conditions, the final choice of the size and number of hoppers or troughs should be based on the first cost of the mechanical equipment for cleaning, as well as on the cost of the bare chamber, and on the operating and maintenance charges of different numbers of cleaning machines. The saving in the cost of foundation work where several shallow grit pits are used may more than balance the extra expense of their more expensive machinery equipment, as compared with the same figures for few but deep pits.

In a few German cities, circular grit chambers have been used, but experience has not developed any favorable opinion of them. In the older type, these chambers were merely large circular vaults with a rack crossing them diametrically, so that they served for coarse screen-

ing as well as settling purposes. Somewhat later, a circular chamber was tried experimentally at Dresden, in which the entering sewage was deflected to the right and the left of a central hollow masonry pier.

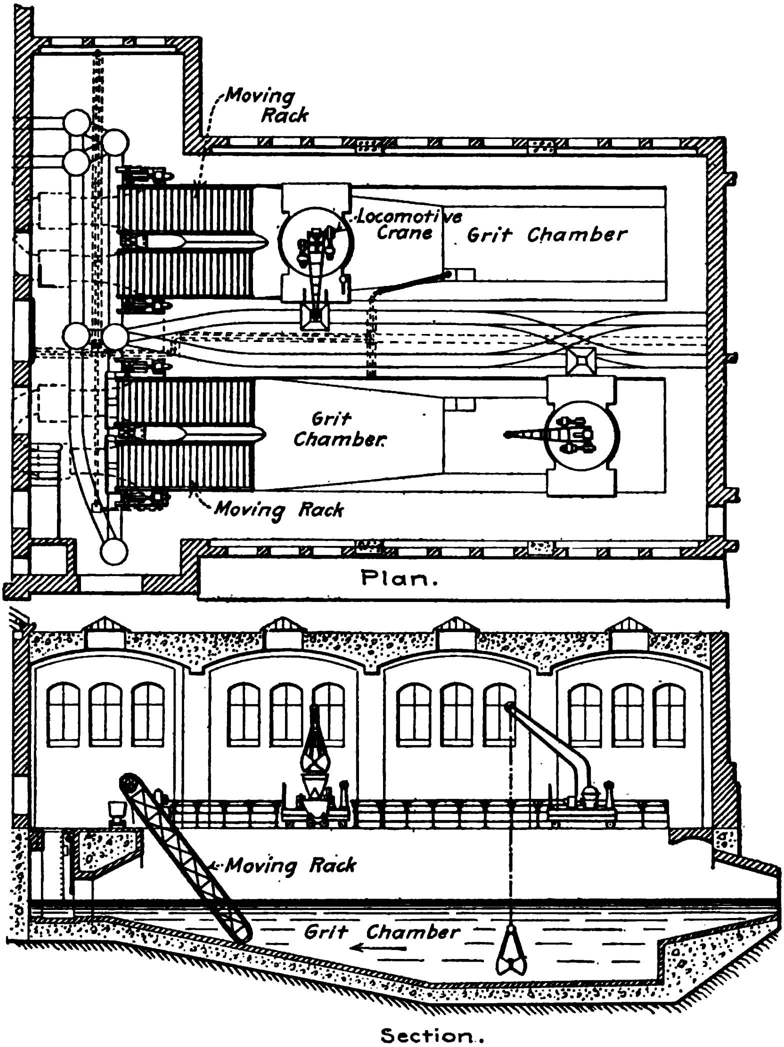


Fig. 46.—Grit chamber on the left bank of the Elbe River at Hamburg.

These two currents followed semicircular paths around the chamber and then met at a point diametrically opposite their place of entrance. Here the sudden checking of the currents where they met was expected

to result in a precipitation of the grit, and its sliding down to a pit within the pier.

Longitudinal Section.—The longitudinal section of American grit chambers has generally been formed by vertical walls and a horizontal bottom. This is also the prevailing section for small chambers everywhere, but sloping ends are employed for hopper and trough bottoms in England and a variety of types have been used in Germany, the latest prominent example being presented by the grit chamber on the left bank of the Elbe at Hamburg, Fig. 46.

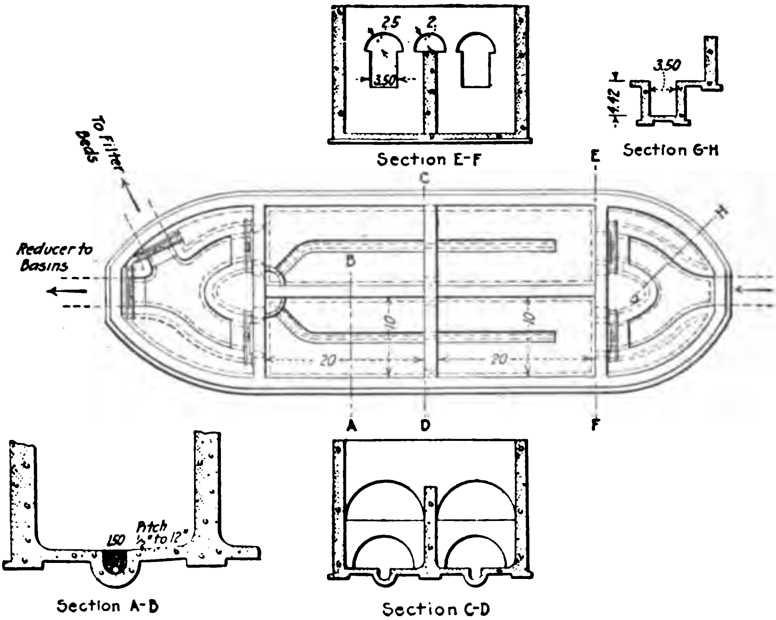


FIG. 47.—Grit chamber at Worcester, Mass.

As the Rochester experiments showed, vertical end walls cause upward currents near their tops, and this fact was the basis of the peculiar longitudinal section selected for the second Hamburg chamber, according to Schmeitzner. Here it was desired to have the heavy solids deposited so far as practicable at the base of the screens, and the bottom was given the longitudinal section illustrated in Fig. 46 to accomplish this. The same section is recommended by Schmeitzner for general use, but with the current flowing in the opposite direction. With such an arrangement and a current velocity adapted to the character of the grit, the heavy sediment will settle out on the sloping bottom and slide slowly down to its toe. The fine grit will settle near the vertical

wall, but the upward current at the top of this wall will keep the lighter organic matter in suspension. Such a chamber is in service at the Charlottenburg pumping station.

Unless it is particularly desired to have the eddy effects, with their vertical currents, at the ends of the grit pit, the inlet and outlet ends of the channels may be flared out so as to change the velocity of flow gradually. The angle of flare which renders the change in velocity most easy is about 13½ deg., but this is too slight for most grit chambers unless used for but a short distance. The change in section in the Imhoff grit chamber, Fig. 51, is unusually gradual, although the whole installation is a compact one. The changes in cross-section in the Worcester grit chamber, Fig. 47, designed by F. A. McClure, City Engineer, are sudden, but the basins are unusually large for the normal quantity of sewage, 15,000,000 gal. in 24 hours, passing through them, each being 40 ft. long, 10 ft. wide and 9 ft. deep, so that any eddy action set up at the inlets and outlets is probably not of much influence on the rate of sedimentation in most parts of the chamber. (See page 309.)

There is a decided difference of opinion among engineers as to the proper depth for grit chambers. Some hold that there should be a pit deep enough not only to hold all the grit which it is desired to intercept in a given time, but also to have above this bed of grit a few inches of practically stagnant water. In other words, they desire absolute freedom from current in the grit pit. Some engineers hold just the opposite opinion, and construct chambers where a current is maintained at all times. Until definite operating information is available concerning tanks of both types, the designing engineer must be governed by his personal preferences toward one form or the other. Fig. 50 is an example of one type and Fig. 51 illustrates the second.

The length of the grit chamber should be governed mainly by the rate at which the grit settles in sewage flowing at the assumed minimum rate. This subject is discussed in Chapter X.

Cleaning Devices.—The design of a grit chamber must provide some method of cleaning it, for if this is not done the purpose of the structure will be defeated whenever the deposit reaches such an elevation that grit is sometimes taken from it and sometimes dropped on it, according to the variations in the current. Accordingly, each of the basins must have either grooves for stop planks or large sluice gates at the ends, so that any one of them can be shut off for cleaning. If only one basin is constructed in the chamber, as in Fig. 50, it must have a by-pass around it. In addition to these details for closing off a basin, it must be furnished with some means for unwatering it, preferably down to the lowest point on the bottom, so as to drain the silt before removing it in case mechanical cleaning is not employed. Methods of drainage are shown in Figs. 50 and 51, but in small chambers it is

not always practicable to provide such systems, and the sewage may be sometimes best removed by a portable diaphragm or centrifugal pump.

Where hand cleaning is practised, the grit may be shoveled into buckets which are raised by a portable derrick and dumped into the carts by which the material is taken to the place of disposal. The transporting and handling of the grit after it has been removed from the chamber differ at every plant, and as the figures of the cost of grit removal usually include both these items, they are not comparable. Inasmuch as rapidity in removal of the grit is desirable, even when the material is handled by laborers, the permanent installation of tracks and pillar crane shown in Fig. 51 has much to commend it. These chambers are most needed during the first part of heavy storms, when their capacity may be taxed to the utmost, and the ability to clean a basin quickly may save considerable damage to a pump or the filling of valuable space in an Imhoff tank with mineral grit which should never be found there if practicable measures to prevent it can be provided.

If the basins are large enough to be cleaned by clamshell buckets, it is desirable to protect their floors with some hard covering, such as a granite-block pavement or steel plates, to prevent the buckets from breaking them. This means of cleaning can probably be employed profitably on many quite small basins, as buckets are now made in small sizes easily handled by portable derricks or gantry cranes. The great advantage of mechanical cleaning for such basins is that they need not be placed out of service while the grit is being removed.

In the usual English form of grit chamber cleaned mechanically, the sides of the chamber incline toward a long sump, so that there is an approximately horizontal run there for the bucket conveyor generally employed to remove the grit. This is indicated in the cross-section of the Manchester chamber, Fig. 48. The head-frame of such an excavator at Toronto is shown in Fig. 56.

The continuous cleaning of grit chambers was discussed by Julian Griggs in *Trans. Am. Soc. C. E.*, vol. lxvii, 1910, page 326, where he stated that some mechanical device should have been provided for keeping clean the chamber of the Columbus, Ohio, main sewage pumping station, because hand-cleaning had proved unreliable. Soon after the station was placed in service, the chamber became filled to the flow line of the sewer, and after that all grit carried by the sewage was swept along into the sump chamber, where it clogged the sewage-level indicators. Part of the grit was caught here and was removed by hand, but probably half passed through the pumps and was finally deposited in the primary septic tanks. There it formed layers over the septic sludge, confined the gases of decomposition and greatly increased the violence of the ebullitions when they passed off.

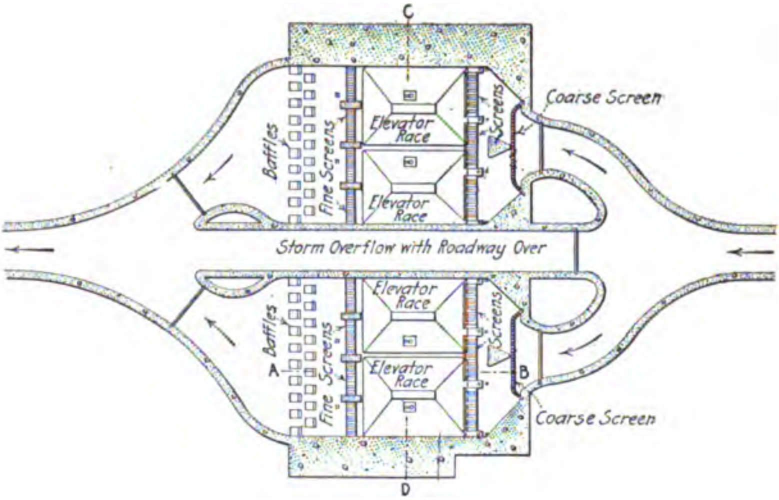
The use of belt conveyors to handle grit after it was removed from the grit chambers did not prove satisfactory in the old Hamburg clarification works. The original installation had a balata belt 397 ft. long, 1.64 ft. wide and 0.28 in. thick, but it was destroyed by the sewage which came in contact with it. For this reason link belts made of galvanized steel plates, such as are used in Cologne, are preferred by some German engineers, although the durability of their joints when grit is transported is questioned. As a rule, small tip cars running on narrow gage tracks are apparently preferred in Germany to any other means of transporting this material.

EXAMPLES OF GRIT CHAMBERS

Manchester.—The grit chambers at Manchester, England, illustrated in Figs. 48 and 49, were put into service early in 1900. The plant is in duplicate, one set on each side of a central storm-water by-pass. One-half of the plant is able to care for the minimum dry-weather sewage, but both parts are used when the quantity of sewage is large. The three sets of screens at the inlets and outlets of the chambers are described in the next chapter. The grit chambers have hopper bottoms and a bucket elevator is carried down into the lowest portion or race of each. The buckets tip their contents into hoppers which discharge into a trough, in which the material is moved along by endless chain scrapers. This plant has handled as much as 50 tons of grit in 24 hours during storms.

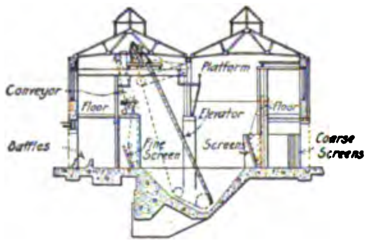
Rochester, N. Y.—The grit chambers of the works for treating the sewage of Rochester, N. Y., are three in number, two for alternate operation at present and the third to meet future conditions. The works were designed by E. A. Fisher, City Engineer, and Emil Kuichling, Consulting Engineer. Only the dry-weather sewage passes through them; the excess flow during storms is conducted around the chambers in by-pass channels, the entrance to which is above the invert of the sewer, so that the heavier suspended matter goes into the chambers as in dry weather. The flow is diverted to the chamber in use by a gate at the entrance to each chamber. There are coarse racks with 2-in. openings at the upper end of each chamber and at the entrance to each by-pass.

The grit chambers have an effective length of 58 ft. and a top width of 17 ft. The sides are vertical to within 7 ft. of the bottom, whence they slope at an angle of 45 deg., resulting in a bottom width of 3 ft. The bottom has a longitudinal slope 3 ft. lower at the entrance than at the outlet, where the total depth is 14 ft. There is a drain pipe at the lowest part of each chamber, for drawing off sewage during cleaning into a well, from which it is pumped back when the cleaning is done. The

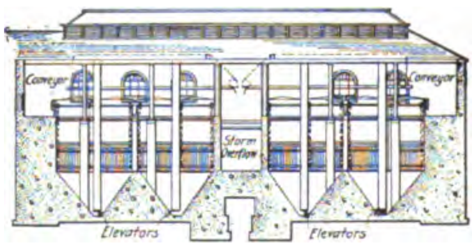


Plan

FIG. 48.—Plan of Manchester, England, grit and screen chambers.



Section A-B



Section C-D

FIG. 49.—Sections of Manchester grit and screen chambers.

grit is removed by power-driven excavating apparatus which can be moved from one chamber to another.

The flow from the chambers is controlled by weirs placed at such a height that for a flow of 50 cu. ft. per second, the depths on the crest will keep the sewage surface below the elevation of the entrance to the by-pass channels. When the dry-weather flow exceeds 50 cu. ft.. another tank is placed in operation.

The storm-water flow is controlled by the back-water in the channel from the grit chambers to the Imhoff tanks. For a flow up to 75 cu. ft. per second the weirs have a free overflow, but as the quantity rises above 75 cu. ft. the sewage surfaces adjust themselves to maintain a flow of approximately 50 cu. ft. for the weir when one chamber is in operation or 37.5 cu. ft. when two are in operation, the surplus passing through the by-passes. The dimensions of the by-pass channels were chosen so that with the maximum depth of back-water in the channel to the Imhoff tanks and two chambers in operation, the by-passes carry the maximum quantity in excess of 75 cu. ft. with a depth of water that gives a submergence of the weir capable of passing 37.5 cu. ft. over each weir. In other words, for the maximum flow the sewage surface in the grit chambers is higher than the surface of the back-water from the Imhoff tanks by an amount equal to the head required to pass 37.5 cu. ft. over each weir.

The sewage surface in the grit chambers varies 1.3 ft. in height between the elevations of maximum and minimum flow, and the velocity varies from 0.3 to 0.5 ft. per second.

Fitchburg, Mass.—Although it is customary to locate grit chambers close to the pumping station or sewage-treatment works of which they are theoretically a part, it sometimes is desirable to construct them at a distance, in order that they may serve an additional purpose which their usual location would render impossible. For example, in designing the intercepting sewer and sewage-treatment works for Fitchburg, Mass., for which David A. Hartwell was chief engineer and Harrison P. Eddy, consulting engineer, a 30-in. inverted siphon was provided on the sewer to the treatment works, in which the sewage was estimated to have a maximum velocity of only 1½ ft. per second during the early years of operation. This velocity was considered too low to keep grit in suspension, and it was accordingly decided to construct a grit chamber on the line of the sewer above the siphon. There are no lateral connections between it and the head chamber of the siphon.

In Fig. 50 are shown the general features of this chamber, which was designed to keep the velocity of flow through it within certain limits, irrespective of the quantity of sewage; in the design account was also taken of the fact that the heavier sand and gravel are washed along the bottom or are carried in suspension in the lower part of the stream.

Small flows up to a depth of 6 in. are confined within a channel of the same shape as the invert of the sewer. When the depth is more than 6 in. the sewage spreads out upon the floor of the grit chamber and the velocity is reduced approximately in proportion to the increase in the area of cross-section of the stream. Diaphragms or dams are placed in the central opening to prevent the creation of currents in the grit chamber below the profile of the invert of the sewer.

The chamber was so designed that in general the velocity would not be in excess of 1.62 ft. per second or less than 1 ft. per second, it being considered that this lower velocity would prevent the deposition of most of the organic matter. The small amount of sediment collecting on the sloping floor of the chamber is washed into the central pit. The length of the structure was based on the time required for the finest sand to be removed from the sewage to a point below the invert of the sewer. The critical size of sand was taken at 0.4 mm. (0.16 in.), as it was believed that any attempt to remove finer material would result in the deposition of quantities of organic matter. As material along the sides at times of high flow have less distance to fall than that at the center, the grit chamber is correspondingly shorter as the sides are approached.

Boston.—In the main drainage system of Boston, designed under the direction of Joseph P. Davis, the provision for intercepting grit is an unusual one. The sewage first passes through cage racks with 1-in. spaces, and is then pumped through 48-in. force mains into a "pipe chamber" at the head of so-called "deposit sewers" nearly a quarter of a mile long, through which the sewage flows at reduced velocity to a shaft leading down to a tunnel under Dorchester Bay. The deposit sewers are two parallel conduits 16 ft. high and 8 ft. wide, the unusual cross-section having been adopted in consequence of daily variations in the elevation of the surface of the sewage in reservoirs on Moon Island, from which the sewage is discharged during ebb-tide. The deposit sewers are dammed at their lower ends to maintain a depth of 8 to 10 ft., in order that the rate of flow through them may be low and a large part of the suspended matter may be deposited in them. The rate of flow in 1913 was about 1½ ft. per second.

Methods of cleaning out the deposits received much attention from the designers of the system. Finally a wooden tank, 50 ft. long, 10 ft. wide and 15 ft. high, was built about 120 ft. from the sewers on the shore of the bay. One end of the tank was connected with the deposit sewers by two 6-in. iron pipes and the other with the chamber around the shaft at the end of the sewers by a 12-in. pipe; in 1901 a different connection was substituted for the 12-in. pipe to the chamber, but the method of operation was unchanged. By means of stop planks the surface of the sewage in the sewers is raised several feet above that

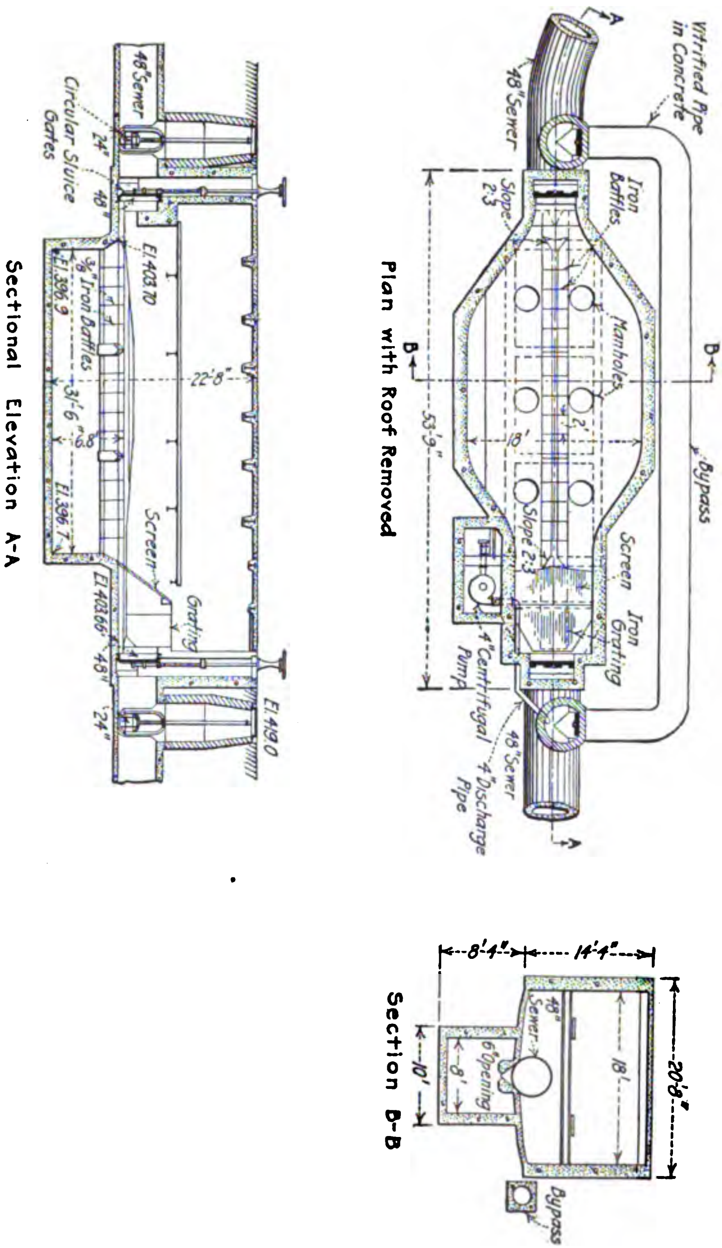


Fig. 50.—Grit chamber on intercepting sewer, Fitchburg, Mass.

in the chamber and circulation is thus established through the 6-in. pipes, tank and 12-in. pipe. The sludge is pushed through the sewers by aprons suspended from rafts or scows, as is done in the sewers of Paris, and is flushed into the tanks through the 6-in. pipes. In 1903, about 8.45 cu. ft. of sludge was removed from these sewers in this way per 1,000,000 gal. of sewage; in 1904, 7.6 cu. ft. and in 1909, 6.7 cu. ft.

Emscher District.—In the Emscher district in Germany, Imhoff employs grit chambers only in cases where the sewage contains considerable sand from the streets, which would fill up the sludge chambers in the tanks forming the leading feature of the treatment plants he designs. The chambers are built in the form of duplicate long, narrow, shallow basins, Fig. 51, with their floors 15 to 18 in. below the invert of

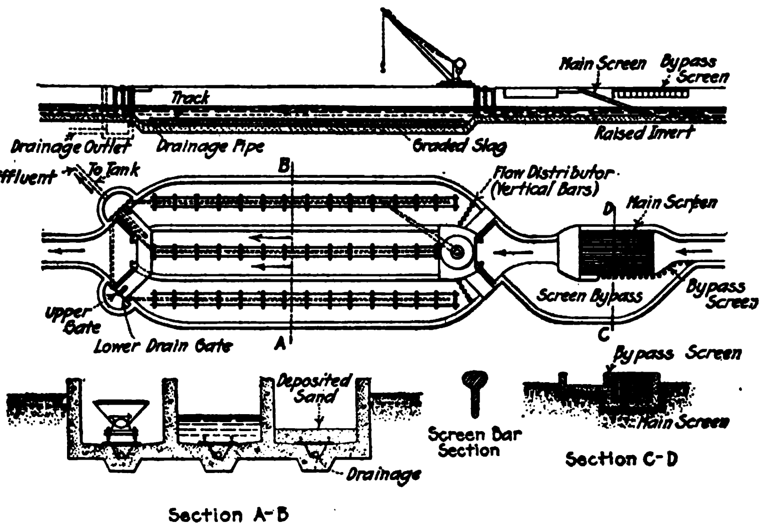


FIG. 51.—Imhoff screen and grit chamber.

the sewers. They are designed to prevent the rate of flow from exceeding 1 ft. per second, and under these conditions nothing but clean sand is deposited. The bottoms of the chambers have tile drains covered by cinders. Every 2 or 3 days, or when the surface of the sand deposit has risen to the invert of the inlet, the sewage is diverted to the duplicate chamber, most of the sewage above the sand is drawn off, and the gate at the outlet of the drains is opened. In this way the remaining water is drained off, and the sand is then shoveled out and used to surface the sludge-drying beds.

Saltley.—The grit chamber at Saltley, on the system of the Birmingham, Tame and Rea District Drainage Board, removed during

1912 and 1913 about 29.2 cu. yd. of grit per 1,000,000 gal. of sewage, according to John D. Watson, Engineer to the Board, who furnished the material for the following notes and accompanying illustrations. The raw sewage contains 577 parts[1] of suspended solids per million.

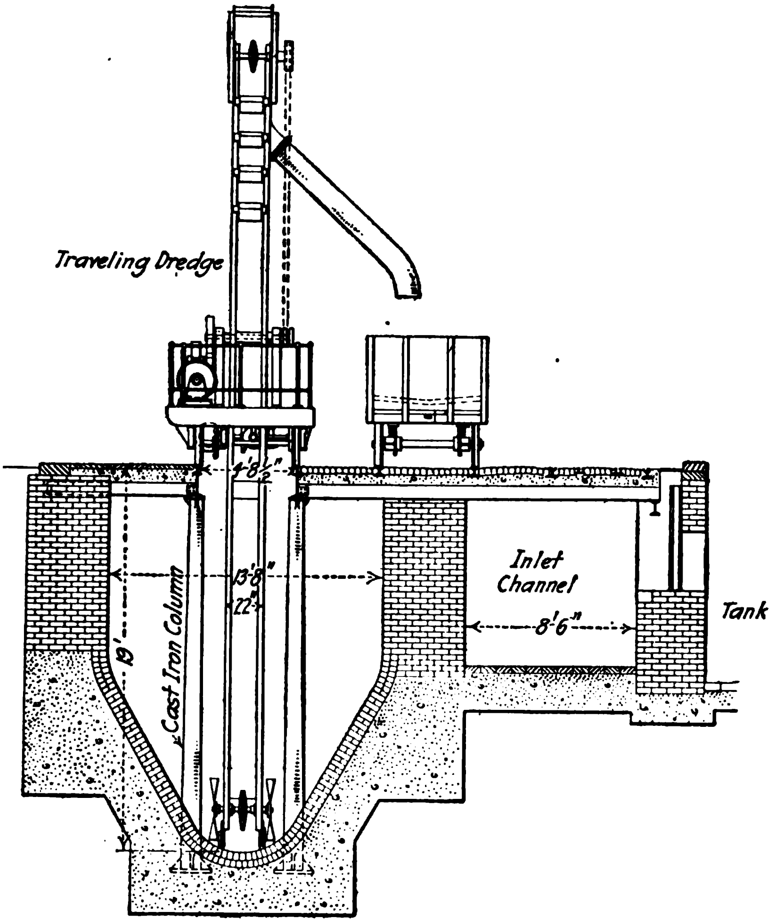


Fig. 52.—Section of grit chamber and dredge, Saltley, England.

The section of the chamber is shown in Fig. 52 and its length is 240 ft. There is an opening in the roof for the entire length of the basin, through which the ladder of the dredge extends. This ladder is 40 ft. long and supported on trunnions about 13 ft. from the top. The

[1] The sewage reaching the works lower in the valley has less solids, 278 parts per 1,000,000 at Erdington, 355 at Cole Hall, 210 at Ashold and 75 at Sutton Coldfield.

buckets are perforated to allow the water to escape, and their contents are discharged through a hopper and chute into a sludge car, which travels behind or in front of the dredge, instead of at one side as shown in Fig. 52, which was the arrangement originally planned. The dredge is operated by a single 15-h.p. motor taking its current from the three overhead wires shown in Fig. 53. The sewage flows through the basin at the rate of about 0.25 ft. per second. The ladder dredge is considered by Watson to be superior to the fixed elevating conveyors used at other works under his charge, since with the latter the breaking of a chain or the renewal of a sprocket wheel necessitates turning the sewage into a

Fig. 53.—Dredge used at Saltley grit chamber.

by-pass and emptying the grit chamber before repairs can be made, while the Saltley dredge ladder can be lifted out of the sewage at a moment's notice.

New Bedford, Mass.—At New Bedford, Mass., the streets are liable to become littered with sand during dry weather, and this sand is swept into the sewers by the storm-water run-off. In order that it shall not enter the submerged outfall sewer, a combined screen and grit chamber was designed by William F. Williams, which has unusual features for cleaning and removing the intercepted sand without offense to those residing in the vicinity. The usual arrangement of grit chambers would, it was feared, cause odors during the cleaning operations, and the material removed would be objectionable for filling low land in the neighborhood. Accordingly it was decided to remove the sand mechanically at frequent intervals, wash the organic matter out of it,

and discharge it on low land. It is proposed to accomplish this by lifting the deposit by ejectors into washers like those used at water filtration plants, and, after it is washed, removing it by other ejectors as sand is handled at some water filters. The plant had not been placed in operation when the information for these notes was furnished by its engineer.

There is a grit chamber on each side of a dry-weather sewer, at this place, the three forming parallel conduits. The screens are located below the chambers. Each chamber is 48 ft. long, 9 ft. wide, and provided with a hopper-shaped bottom running down 6 ft. below the elevation of the sewer invert. The main floor of the building over these structures is 22.8 ft. above the hopper bottom and has an opening over the center line of each hopper, with cast-iron cover plates, through which the grit can be removed by a grab bucket and traveling crane if the sand-washers fail. Two centrifugal pumps have been provided for pumping out the chambers after closing the gates at each end. After the sewage has been drained off, it is proposed to have men push the grit to a vertical ejector, which will lift it into one of two sand washers in a room over the dry-weather sewer. Each washer is 3 ft. 8 in. in diameter and 4 ft. high, including a conical bottom. The clean water used in ejecting the sand also cleans it in the washer and then escapes through the closed top of the washer into pipes discharging it into the sewer. There is a glass window in the side of the washer to enable the attendant to observe the operations within, and there are two water connections to help wash and move the sand. The washed grit falls into a Nichols ejector of the type used at the Philadelphia water filters, which forces it to the low ground to be filled.

It is expected, the authors are informed, that the grit chambers will be used only during rainfalls of sufficient intensity to carry sand into the sewers. Experiments will be made to determine how long a period must elapse after rain begins before it will be necessary to divert sewage into the chambers, and similarly how much time must elapse after rainfall ceases before shutting off the chambers. It is hoped that it will be possible to remove the grit as fast as it is deposited, but this may not be practicable.

Worcester, Mass.—Ordinarily only one chamber of the Worcester grit chamber, page 298, is in service. Both inlet and outlet gates are wide open. With a small flow of sewage, sludge high in organic matter is deposited unless the chamber is nearly full of grit. The grit settles at the inlet end first, and the accumulation gradually extends toward the outlet end. The sludge is displaced by the advancing deposit of grit and it has proved best to defer cleaning the chamber until the entire accumulation is quite solid. The deposit would be less offensive if the chambers were smaller and more numerous. The capacity of the grit chamber is none too great during storm-water flow.

CHAPTER IX

RACKS, CAGES AND SCREENS

Among the objects accomplished by screening the following are prominent:

1. The protection from injuries and clogging of appliances for conveying, pumping and treating sewage and sludge.
2. The prevention of unsightly matters floating in tanks of treatment works or in bodies of water into which unsettled sewage is discharged.
3. The reduction of the amount of sludge settling on the bottom of slow-moving bodies of water and liable to cause offense by decomposing.
4. The prevention of heavy and extremely tough floating scum on the surface of septic and other tanks.
5. The removal, so far as possible, of the fine suspended matter in addition to the coarser matters mentioned above, either as a final treatment or as a preliminary one to assist further purification processes.

As a temporary expedient, for service while developing more complete methods of treatment and building the plant for applying them, screening offers the advantages of low cost and considerable efficiency. For example, the discharge of raw sewage into Flushing Bay, N. Y., is prohibited by law. It was very desirable in the fall of 1911 to place in service as soon as possible a small sewerage system in the Borough of Queens, New York City, naturally discharging into that bay. This system was planned to discharge into an intercepter leading to a distant outlet, but this intercepter was not likely to be finished at an early date. It was accordingly determined to install a temporary fine-screening plant in the main sewer and discharge the screened sewage into the bay, which was permissible. The main sewer is a 9-ft. circular structure, and the screens were placed in its lower half. They comprise a rack of $\frac{3}{8} \times 1\frac{1}{2}$-in. bars, a screen of $\frac{5}{16}$-in. galvanized iron wire of 1-in. mesh, one of No. 8 galvanized iron wire of $\frac{1}{2}$-in. mesh, one of No. 14 gage brass wire of $\frac{1}{4}$-in. mesh, and one of No. 18 gage wire of $\frac{1}{8}$-in. mesh. The whole expense of the installation, including frame house, plumbing, equipment and furnace for refuse, was $2538.

Classification of Screening Plant.—There is no agreement among engineers as to the size of the opening which is on the boundary between coarse and fine screening. In a general way, the tendency of the present time seems to be to regard an opening of $\frac{1}{4}$ in. as the maximum limit for fine screening. An opening of this size will pass a large amount of

coarse suspended matter, but as a rack or screen with such openings continues in service, the refuse which collects on it is likely to reduce considerably the actual size of at least some of the apertures, so that the screen will hold back a part of the suspended matter which would pass through a ¼-in. opening theoretically.

Screening can be divided according to its purpose into, first, fine screening which has certain of the elements of a treatment process, and, second, coarse screening, which is not primarily intended to be such a process. The efficiency of screening for the first purpose depends upon the size and quantity of the suspended matter and the screen openings.

"As Bredtschneider has shown, the average diameter of the sludge-forming matters is generally less than 1 mm. (0.04 in.). If, therefore, the suspended matters are not massed together in the sewage, but reach the screen in a finely divided condition, as, for example, in the case of very long sewerage systems with considerable fall and many bends, and also following pumping stations, or in case water-closets with narrow outlets are used (diameter less than 10 cm., 4 in.) then no satisfactory results can be obtained even with the finest screen yet built. It is, of course, possible to construct screens with still smaller open spaces than above mentioned (1¼ mm., 0.05 in.), but it is uncertain whether they will prove practicable, as the fine wires or sheets of which they must be constructed would offer but little resistance to the acids in the sewage and would be quickly broken by the cleansing brushes." ("Clarification of Sewage," Schmeitzner, Kimberly's translation, page 47.)

"These data indicate that most of the suspended matter in sewage is usually in a very fine state of division, or capable of passing through a sieve having meshes 0.02 in. square; that most of the solid fecal matter found in the liquid can be arrested on a sieve having meshes 0.25 in. square; and that a sieve with meshes 0.10 in. square is probably fine enough to extract from the sewage all suspended matter of appreciable magnitude, and especially such as is likely to become offensive under the conditions herein proposed for the disposal of the sewage of Rochester." ("Notes on Sewage Disposal," Kuichling, 1910, page 17.)

The following definitions are introduced here to avoid confusion as to the meaning of certain terms. The word "rack" signifies screening apparatus consisting of parallel bars. The word "grating" means screening apparatus composed of two sets of parallel bars, the sets intersecting at right angles. The word "screen" indicates either wire cloth or a plate of metal perforated with holes of small size. By "wire racks" will be designated the few screens consisting of parallel wires stretched tightly between frames. If the racks are stationary they will be termed "fixed," and if they are capable of motion they will be termed "movable"; the same classification will be applied to screens.

Large pumping plants are sometimes protected by racks with openings so large that they seem valueless in checking the passage of objects

likely to injure the machinery. For example, the first rack at the Manchester, England, treatment works, (Fig. 49), is composed of 1 × 4½-in. bars, 6 in. apart. It is needed, however, to catch large timbers and similar heavy floating objects which, if thrown against a rack of lighter bars, might distort them. While there is no need of these very coarse racks in most cases, it is a good plan where a combined system of sewerage is used to consider very carefully whether the construction of the street inlets is such that large pieces of timber may be carried into and at a fairly high velocity through the sewers. There are also in many combined systems open brook channels tributary to the sewers. Such channels usually receive considerable quantities of refuse and trash some of which may be very large, such as barrels, boxes, cans, lumber, railway ties, brush and trees.

Fibrous Material.—If the sewage contains cotton waste and similar materials which interfere with the operation of pumps, particularly small centrifugal pumps, racks finer than are otherwise necessary are likely to prove useful. Some engineers consider that no attempt to screen sewage containing waste or to use centrifugal pumps smaller than 6 in. for handling such sewage, is advisable, unless the flow is so large that the cost of attending to the screens, per million gallons passed, is not burdensome. The cost of keeping clear racks and pumps smaller than 6 in. may prove greater than the additional fixed charges on pumps of larger size.

The effect of fibrous material on screens was shown in 1900 in experiments at Leeds, England, by Harding, Hewson and Harrison. A screen of zinc perforated with ⅛-in. holes formed a trough 3 ft. wide, about 12 in. deep and 40 ft. long. When raw sewage was admitted to this, the liquid at first fell through the perforations in a fine rain, the stream reaching only a few feet forward. Solid matters were washed ahead to the edge of the flow, and the screen might have proved entirely satisfactory but for the large quantity of wool fiber brought down in the sewage, which gradually choked the perforations at the entrance end and caused the flow to extend along the trough. The trough had to be brushed clean from time to time, in order to keep the flow under control. The authors have made effective use of inclined perforated plates in treatment plants for industrial wastes containing wool scourings and waste water from cloth washing.

Location of Racks.—The best position for racks depends to some extent upon local conditions, the type of grit chamber, and the character of the machinery, if any, used in removing the grit. Where the grit chamber is large, to retain much coarse organic matter, the work of cleaning the racks will be much less if they are placed at the outlet of the chamber than at its inlet. An objection, which is sometimes serious,

to placing racks above grit chambers is that when partially clogged the racks cause the sewage to back up in the sewers, forming deposits there rather than in the grit chambers. As the sewage passes through racks it is given an irregular, eddying motion likely to reduce the sedimentation in a chamber immediately behind the racks, which is another reason for placing the racks in the outlets of such chambers.

In the screening plant shown in Fig. 49 the sewage first passes through a protecting rack with 6-in. openings, and then through a rack 37 ft. long, in three independent sections. It has 3/8-in. bars with 1 1/4-in. openings. The sewage then passes into a grit chamber and finally through a third rack, of 3/8-in. bars with 1/2-in. openings, constructed in four independent sections.

DESIGN OF FIXED RACKS

Length.—Racks and screens are generally placed at right angles to the axis of the channel they cross. Their length should depend on the amount of sewage to be screened, the size of the openings, and the permissible loss of head. If only a narrow channel is available, the rack may be curved in plan or arranged along the sides of an angle, so as to make it longer than a straight rack. Another way to secure a large screening area is to make the slope of the rack quite flat. Boston experience indicates that the area of the racks should be at least 150 per cent. of that of the channel leading to them.

Width of Openings.—Cage racks with openings of about 1 in. have become a standard form of construction in the vicinity of Boston for screening large quantities of sewage. The size of screen openings necessary to protect centrifugal pumps with closed impellers is given in Table 168 of Volume I. The size of the openings in racks guarding treatment works is gradually becoming somewhat standardized at 1/2 to 1 1/2 in. It is possible that, under some conditions, screening through racks of not over 1/2-in. opening may prove useful, particularly in preventing thick and sometimes troublesome scum.

The practice of F. A. Barbour is based on experience which he has stated as follows:

> "The screen bars are round, 3/4 in. in diameter, and spaced 1 3/4 in. on centers. This was found to be not close enough for the operation of 6-in. centrifugal pumps of the usual type, and a screen of closer mesh was subsequently placed inside the cage. For more recent work it has been found that square are better than round bars for fine screens, and a cage of 5/8-in. bars spaced 1 1/4 in. on centers is about right for small pumps." (*Jour. Assn. Eng. Socs.*, 1905, vol. xxxiv, page 42.)

An investigation of the effect of the cross-section of bars on the discharge through a rack composed of them was made at Cornell Uni-

versity by Henry Ryon. (*Cornell Civil Engineer*, 1910.) The slats measured 3 × 0.5 in. and were spaced to give 0.5-in. openings. If the discharge of a rack of rectangular bars is taken as 1, the discharge with the downstream ends of the bars pointed was found to be 1.026; that with the upstream ends pointed 1.214, and that with both ends pointed 1.272. The shape of bar adapted to give the maximum flow through a rack is not necessarily the best to intercept screenings.

With screens and racks having openings smaller than ½ in., there are three features to be considered particularly: first, the danger of unduly raising the sewage level at the rack by the clogging of the openings with screenings; second, the arrangements that must be made for the disposal of the large quantity of refuse from such fine screens; and, third, the cost of labor or machinery required for cleaning. The first point is important because if screening devices become so clogged as to cause a pooling of sewage in the sewers leading to the grit chamber, then troublesome deposits may be formed in the sewer. As for the second point, there is a probability that materials removed by fine screens from sewage, even fairly fresh sewage, will give off unpleasant odors if not speedily taken care of.

Duplication of Screens and Racks.—Where the screening devices have an important function which must be performed regularly in order to insure the satisfactory operation of the works, it is necessary to provide duplicate apparatus so that one unit may be in service while the other is being repaired or cleaned. This duplication may be provided by placing a movable rack or screen behind another in the same channel or by providing two channels side by side, each containing one or more screens or racks, either fixed or movable. The preference is generally given to two channels, because where there is but one an accident to a screen may render it difficult for the other to serve its purpose.

Designers should also give attention in planning such installations to the danger that always exists where the American type of vertically movable racks is employed for screening. If there are two racks crossing a channel, and the first is lifted for cleaning, the second collects all the rubbish and the pumps or treatment works are not affected. But if it becomes necessary to clean the second or rear rack, it will sometimes be observed that when this is raised, no matter how carefully it may be done, a considerable quantity of refuse drops from the rack into the sewage and is swept along. Bottom plates have been provided to hold this loose refuse as the racks are raised, but they answer their purpose unsatisfactorily.

Head on Racks.—In preparing the plans attention should be given to the increase in head of the sewage likely to occur as a result of clogging of the racks.

"It would be a mistake to assume that in consequence of the dimensions

of the rack only substances of 15 and 10 mm. (0.6 and 0.4 in.) and greater are intercepted, for much smaller particles, and sometimes even very small particles, are intercepted on the racks. Paper collects on the bars and the cross-section between the separate bars becomes steadily smaller. Occasionally the rack has temporarily become practically impervious like a solid wall. On this account it is desirable to design such racks to carry a considerable head of water. While the sewage backs up in front of the racks an average of 20 cm. (8 in.), it rises during heavy storms to 70 and 75 cm. (27½ to 29½ in.). ("Kanalisation der Freien und Hansestadt Hamburg," Merckel, page 197.)

As a preliminary step in determining the probable effect of a rack on the elevation of the sewage in the channel above it, a computation may be made by the following formula:

$$\frac{1}{2g}(V^2 - v^2)\frac{1}{0.7} = h$$

FIG. 54.—Waste-weir at rack of an Emschergenossenschaft plant.

where V represents the velocity in feet per second through the openings of the rack, v represents the velocity just before the sewage reaches the rack, g is the acceleration of gravitation, 0.7 is a velocity coefficient suggested for this use by Frühling, who proposed the formula, and h is the head in feet due to the rack.

There is a considerable variation in the importance attributed to keeping racks clean. In some small separate systems, where no storm water enters the sewers, hardly any attention is given to the removal of screenings. In some large stations, like those of the Boston Metropolitan District, the removal of the screenings is occasionally an arduous

task. There is apparently no way to foresee exactly what the conditions will be, and it is the designer's duty, therefore, to make the racks and their supports strong enough so that in case they should become clogged and the sewage dammed up above them, they will be sufficiently strong to resist the extra pressure until the condition is discovered and relieved. If the sewage is likely to back up and place a sewer leading to the screens under a head, the designer may be justified in either installing an automatic alarm apparatus which will give notice, by ringing a bell as at Dresden, or otherwise, of any dangerous rise of the sewage, or providing a by-pass with a waste weir of sufficient length to prevent the sewage rising above a certain height, as shown in Fig. 54.

Another consideration, and one which may often be more important than the danger of surcharging the sewer, is that of causing deposits in the sewer above the screens. It is necessary to give constant attention to the problem of keeping the sewage moving steadily and swiftly through the sewers that it may not become stale or even septic through sluggish flow or prolonged contact with decomposing sludge. Hence racks should be kept reasonably clean.

R. S. Lanphear found at Plainfield, N. J., where the rack openings are ½ in., that the racks became so clogged between 5 and 9 a.m., when they were without attendance, that the loss in head amounted to 6 to 9 in. This much exceeds the loss during the hours of regular cleaning. At the Dresden screening plant, with disk screens having 0.08 × 1.2-in. slots, the lost head is 4 to 24 in. and at Marburg, where the rack has 0.06-in. openings, the loss is 10 in.

Slope of Racks.—Racks are often constructed in a vertical position, although it is then difficult to clean them satisfactorily; the advantage of such a position is that the racks, if movable, can be lifted out of place and put back with minimum trouble, and furthermore, that the vertical arrangement is the most economical of ground area. Inclined racks are more easily cleaned and they also offer a larger area to the sewage, thereby reducing the velocity of flow of the sewage through their openings. There is no agreement among engineers concerning the slope of these inclined racks, but the usual inclination is somewhere from 30 to 45 deg. from the vertical. With racks of moderate size, the greater the inclination the more easy the work of cleaning.

In a few German plants, where the inclination of the racks is very great, their length has been reduced, in order to facilitate cleaning, by supporting their feet on a low transverse wall built up on the invert of the channel. Where such a plan is followed it is manifest that care should be taken to design the wall so that as little sludge will accumulate in front of it as possible.

Details.—Some means for removing the screenings expeditiously should be provided. They may be raked over the top of the rack, which

may be curved or bent, as shown in Fig. 55, to enable the rake to pass freely backward at this portion of the rack into a transverse trough or on to a platform. In some cases the bars of the rack are extended upward to such a height that the screenings are raked over their top directly into a small car running on a platform or track behind the top of the rack. At the sewage pumping station at Charlottenburg, for instance, the rake is hauled upward by a chain passing over a pulley and discharges the screenings in the way described into a small dump car. As the screenings are likely to cause considerable offense they must be removed as rapidly as possible, and are usually buried, burned, or sent to a distant dump. Some attempt has been made to utilize them as

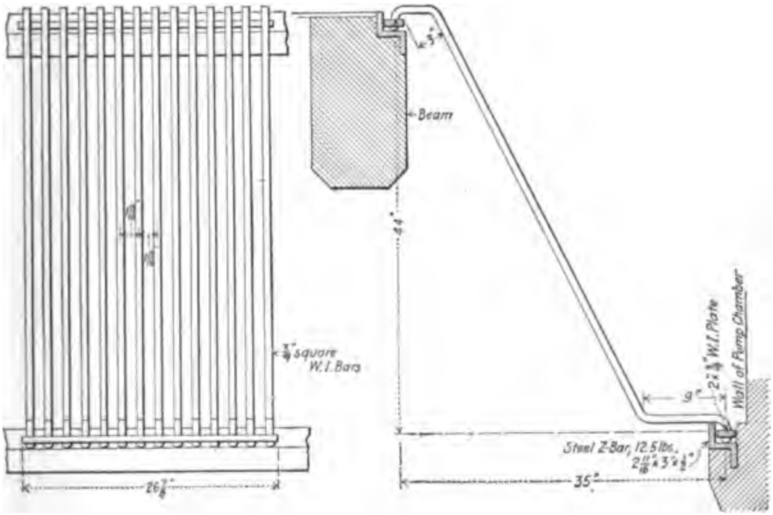


FIG. 55.—Rack used at North Attleboro pumping station.

fertilizer in Germany, but apparently with little success. The incineration of the screenings may be conducted in a special crematory, as at Manchester, England, or in the furnaces of the boiler plant furnishing power for the sewage pumping works, as in several plants of the Boston Metropolitan System.

In detailing the racks it is desirable to keep the supports and spaces of the bars as far from the front of the racks as possible, in order that the tines of the rakes may not be caught as they move along the openings. In the case of small racks, like that at North Attleboro, Fig. 55, designed by Barbour, which protects the inlet to a pump well, support of the bars at the top and the bottom is all that is necessary. Where long bars must be used, however, it may prove desirable where

stiffness is desired and racks of considerable length are necessary, to employ bars with ears or projections at their back, through which the supporting rods of the framework may be passed, as shown in Fig. 319, Volume I. In large plants, the working platform and the entire screening chamber should be designed so they can be cleaned with water from a hose.

Attention should be given to the details of the side and bottom of the rack, for it is by no means uncommon to see well-designed racks surrounded by openings between the steel and the masonry, so large that sewage passes through them unscreened. This is particularly important where the openings between the bars are under an inch, for larger openings at the sides are likely to remain free long after the rack proper has become somewhat clogged and an undesirable amount of coarse material may be swept by the rack in this way.

Another detail to be studied is some provision for closing the channel above the rack, in case the latter must be repaired. In all but very small installations grooves of some type are usually provided for the insertion of stop planks, but frequently nothing of this sort is furnished in connection with small plants, reliance apparently being placed upon closing the channel with sand bags or some other temporary expedient.

MECHANICALLY CLEANED RACKS

An important installation of machinery for cleaning fixed sewage racks was put in at Toronto, Ont., by Ham, Baker & Co., Ltd., in accordance with the requirements of C. H. Rust, while city engineer. Its general arrangement of sheaves and moving belts is like that shown in Fig. 49. There are 6 racks, 2 in chambers 33 ft. deep and 4 in chambers 14 ft. 3 in. deep. Each rack is 5 ft. 8½ in. wide and made up of bars 10½ ft. long and ½ in. thick, with ½-in. openings. The bars to which the malleable cast-iron tines are attached are carried by endless chain belts which pass over sheaves in head frames shown in Fig. 56. Arrangements for adjusting the tension of the chain belts are provided and a revolving cleaning bar with 4 rows of teeth automatically frees the tines from refuse and rubbish. This falls on a sloping tray which discharges it into a screw conveyor. The makers of the apparatus advise adjusting the speed so that the whole surface of the rack is cleaned once a minute. The racks remove 10 to 15 per cent. of the settling solids, according to Commissioner Harris of the Department of Works. The same screw conveyor which removes the screenings also receives the refuse from a bucket elevator running down to the bottom of the grit chamber in front of the screens. The head frame of this elevator is shown in Fig. 56.

The first important installation of this type was at Manchester,

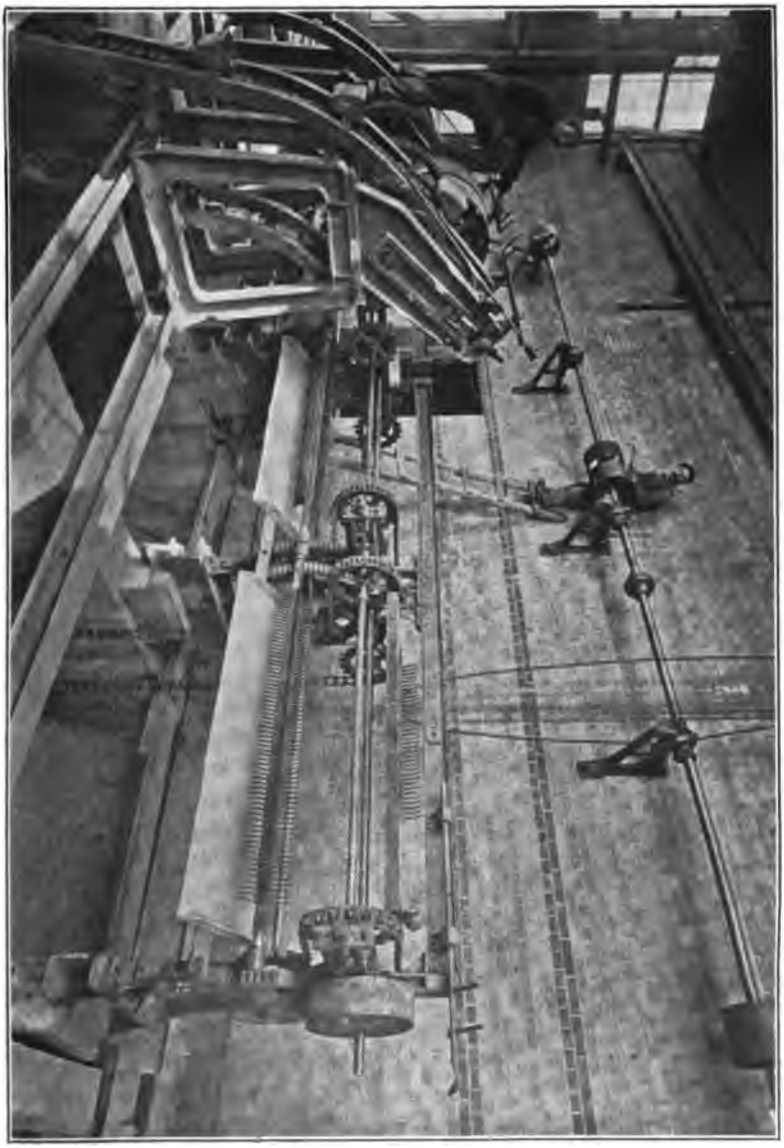


Fig. 56.—Headframes of rack-cleaning apparatus and grit elevator, Toronto.

England, in 1899, in connection with grit chambers built at the same time, Fig. 49. The operation of this apparatus, like that at Cologne, described later, was not smooth when examined by the authors. The jerking motion of the chain belts threw material from the teeth back into the basin, so that eventually it had to be raised again, which reduced the operating efficiency of the apparatus.

A mechanically cleaned rack built about the same time as that at Manchester, and noted for its unusual slope, the top being farther upstream than the toe, is in use at Clichy on the sewerage system of Paris. The racks have openings of 25 mm. (1 in.) and are at the exit of a sedimentation basin 15 meters (49¼ ft.) wide and 60 meters (197 ft.) long. This unusually large basin was provided to remove street refuse carried by the sewage, which otherwise would not only injure the pumping machinery but also tend to clog the fields where the sewage is used for irrigation purposes. The endless chains which carry the rods holding the cleaning teeth are on the downstream side of the rack, and consequently the sheaves at the bottom are in screened sewage, for which some advantage is claimed by European engineers. On the other hand, the teeth have to project through the rack and for some distance on the upstream side, in order to rake up the refuse, and the latter is moved upstream against the current, owing to the inclination of the rack, and consequently there is a somewhat increased pressure tending to drive it through the openings.

One of the leading German examples of mechanical cleaning apparatus in connection with fixed racks was used at Cologne until replaced by disk screens of the Riensch type. The sewage first passed through a rack with 20-cm. (7.9-in.) openings which was inspected daily and cleaned by hand when necessary. It then passed through a rack with 20-mm. (0.8-in.) openings inclined downstream, like American racks, at an angle of about 45 deg. Afterward it passed through another rack of the same general type except that the openings were 3 mm. (0.12 in.). The cleaning was done by brushes attached to horizontal bars carried by endless chains. These chains passed over sheaves at the bottom and top and the cleaning was done on the upstream side of the rack. The upper sheaves were capable of adjustment so that the chains could be maintained under a proper tension without difficulty. There was no trouble with the operation of the lower sheaves in the unscreened sewage. The refuse was pushed up the racks by the steel brushes to an apron at the top, over which it fell upon a belt conveyor. The brushes were cleaned by being rubbed over a comb attached to the head frame carrying the upper sheaves. It is stated that from 5 to 6 h.p. were required to drive this cleaning apparatus.

MOVABLE RACKS

There is a number of types of movable racks, although but one has been employed to any extent in the United States. This is the basket or cage rack, consisting of a plate or grid bottom and three sides of bars or rods, the fourth side being the opening which fits across the channel through which the sewage passes. These cages are raised to be cleaned.

Cage Racks.—The first large American screens of this type were designed for the Boston Main Drainage Works and went into operation January 1, 1884. Later screens for the Massachusetts Metropolitan

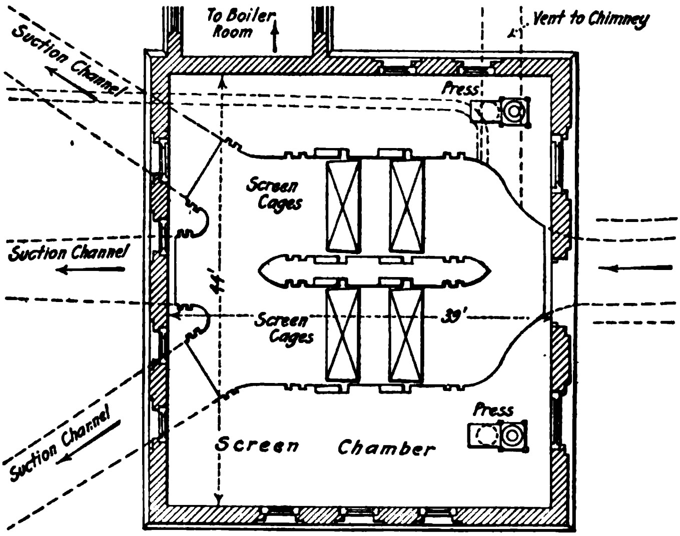
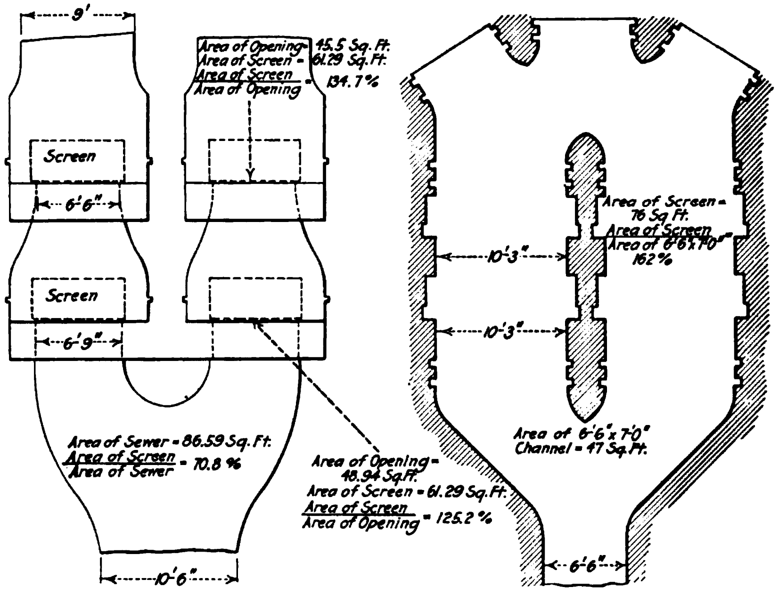


FIG. 57.—Screen chamber at Ward Street station, Boston.

Sewerage District were designed along the same lines. These installations consist of a screen chamber extending from the level of the invert of the sewer to the surface of the ground and surmounted by a superstructure for housing the cleaning and hoisting apparatus. Fig. 57 shows such an installation at the Ward Street Pumping Station of the Massachusetts Metropolitan Sewerage District. Each screen consists of a cage provided on three sides with vertical ¾-in. bars, with an open space between bars of 1 in. The front of the cage, facing the incoming sewage, is open and the bottom is made of iron perforated with holes to allow the water to drain off as the cage is hoisted. Each cage is about 9 ft. square by 3 ft. 6 in. deep and is similar in appearance to a passenger

elevator. After the works had been in operation for a short time and before the official acceptance tests of the pumps, the builders objected to the amount of suspended matter coming through the screens. For this reason a second row of bars was staggered behind the first. These are so placed that the actual open space as measured diagonally between bars is still about 1 in. One of the screens at Nut Island also has a similar second row of bars. At Deer Island and East Boston, this second row of bars was omitted, as it was felt that with the centrifugal pump installations at these places such care in keeping out fine material was not necessary.



Calf Pasture Screen Chamber Ward Street Screen Chamber

FIG. 58.—Comparative screening areas at two Boston screen chambers.

The cages are balanced by counterweights and are manipulated by small reversing engines attached to the frame supporting the cages. At the Calf Pasture Station the channels leading to each set of screens are provided with large hydraulic-operated sluice gates, so that either can be shut off and but one set of screens used at a time, which has been the usual practice. At the Ward Street Station, however, the gates are dispensed with and both channels used regularly. Grooves are provided in the masonry for the insertion of stop planks in case it is necessary to shut off one channel for any reason. Considerable trouble has been caused at the Calf Pasture Station by débris forcing its way

through the screens, which is attributed largely to the relatively small proportion of screen opening to sewer area, particularly when only one channel is being used. The débris clogs the screens rapidly, and often before cleaning a head of water has been built up sufficient to force material through the bar openings. Fig. 58 shows the relation of screen opening to area of sewer at the Calf Pasture and at Ward Street. At

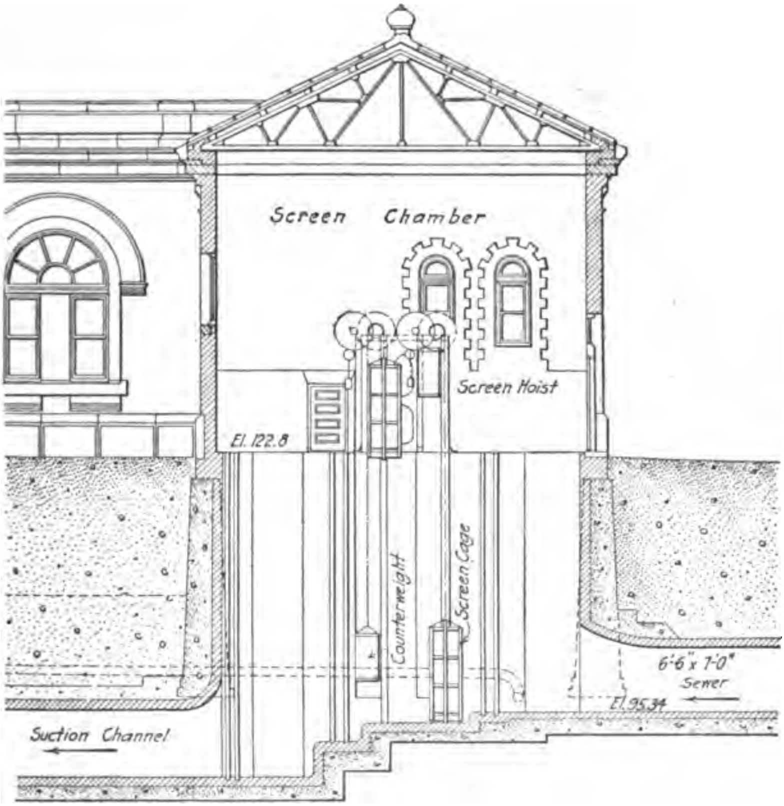
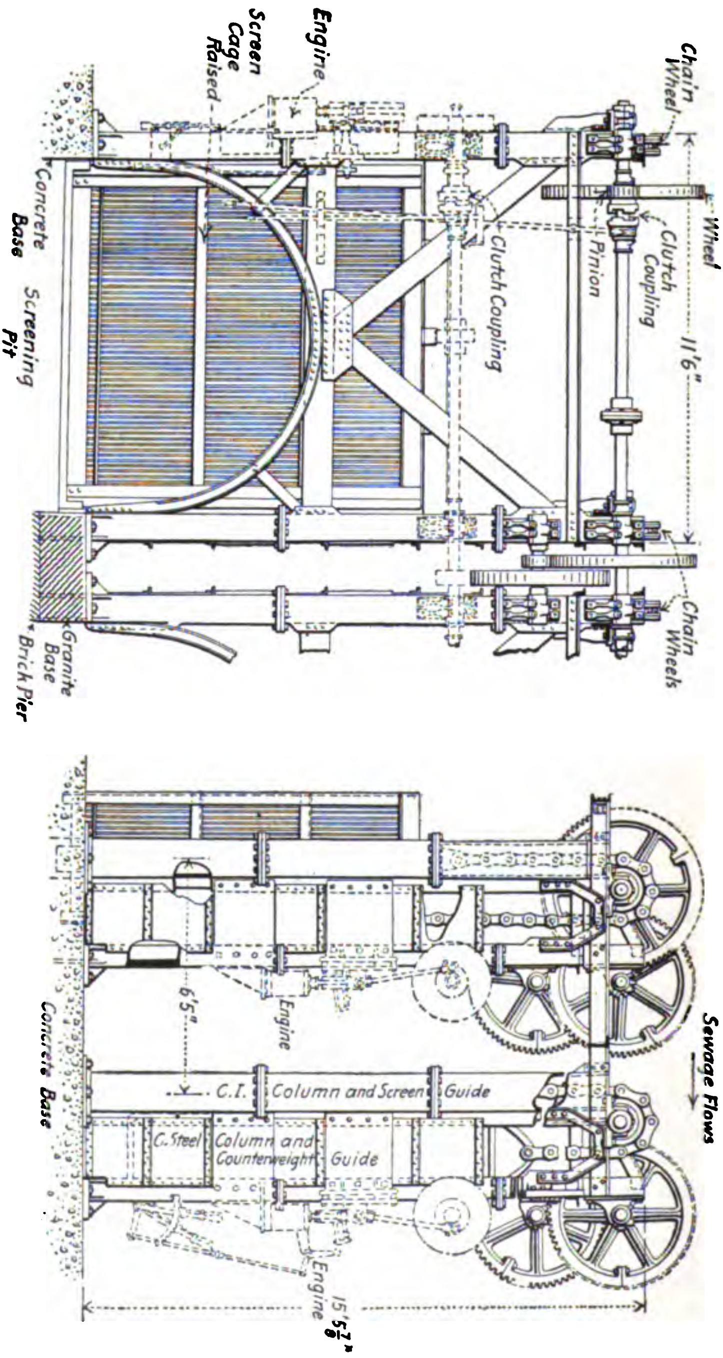


FIG. 59.—Sectional elevation of Ward Street screen chamber.

the former place, with only one set of screens in use, the ratio of screen area to sewer area is 70.8 per cent., and with both sets in use is 142 per cent. At the Ward Street Station the ratio of screen opening to sewer area is 162 per cent. with one screen and 324 per cent. with both screens in operation. Even under these latter conditions, it is said that in times of storm the screens need constant attendance and cleaning to keep them sufficiently free of clogging material.



Elevation Looking Up Stream. Side Elevation.

FIG. 60.—General features of screening machinery, Ward Street station, Boston, Mass.

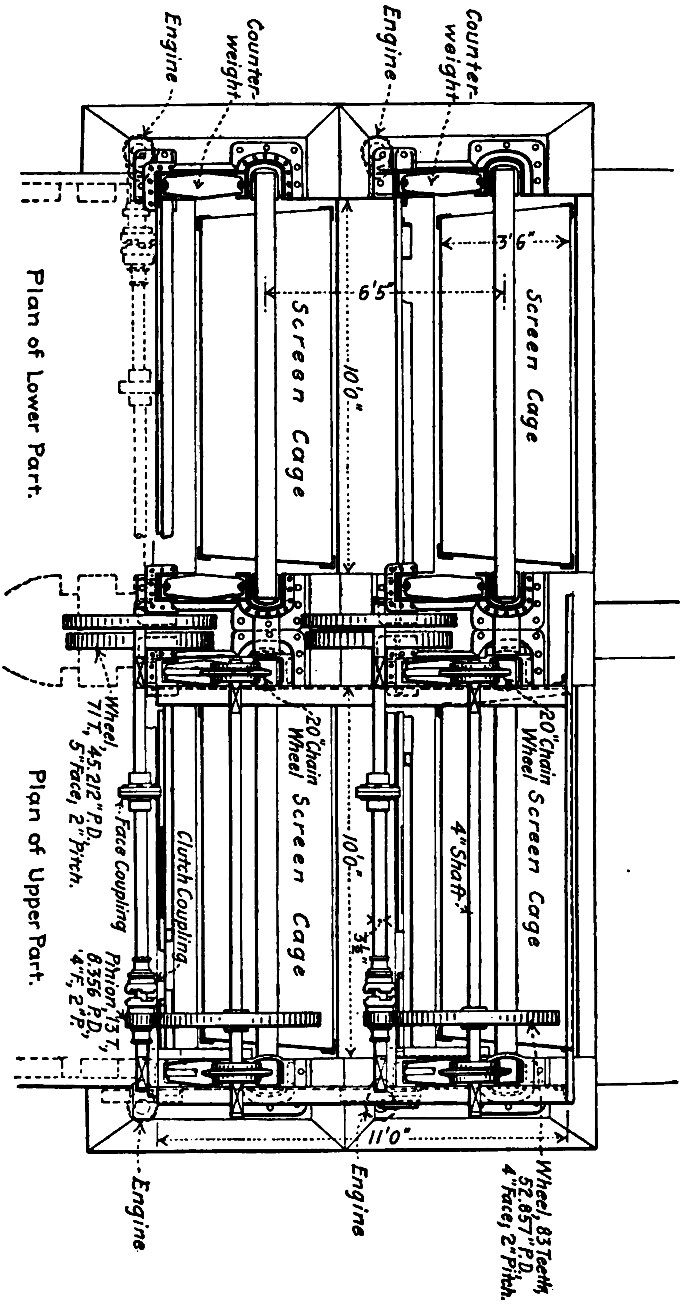


FIG. 61.—Plan of screens and hoisting machinery, Ward Street station, Boston.

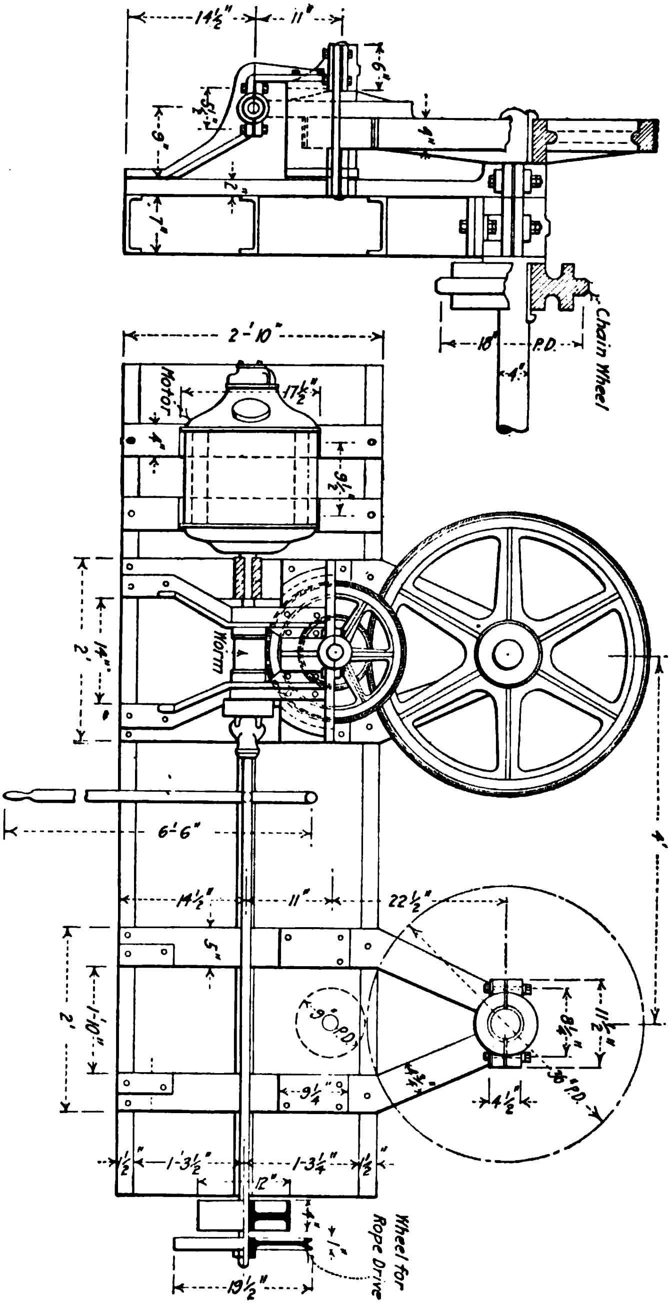


FIG. 62.—Electric drive for cages at East Boston screen chamber.

Each set of two screens is provided with two engines, and by a system of clutches either or both engines may be used for hoisting either of the cages, Figs. 60 and 61. By a system of reducing gears the power is transmitted to chain wheels located directly over cast-iron guides on either side of the cage. These wheels are rigidly attached to a horizontal shaft located over the center of the cage, thus insuring, during hoisting, a positive and equal movement for each side of the cage. There is a 2-in. clearance between the edge of the wall and the cage.

The hoisting apparatus has been considerably simplified at the Deer Island and East Boston Stations by the substitution of a worm direct-

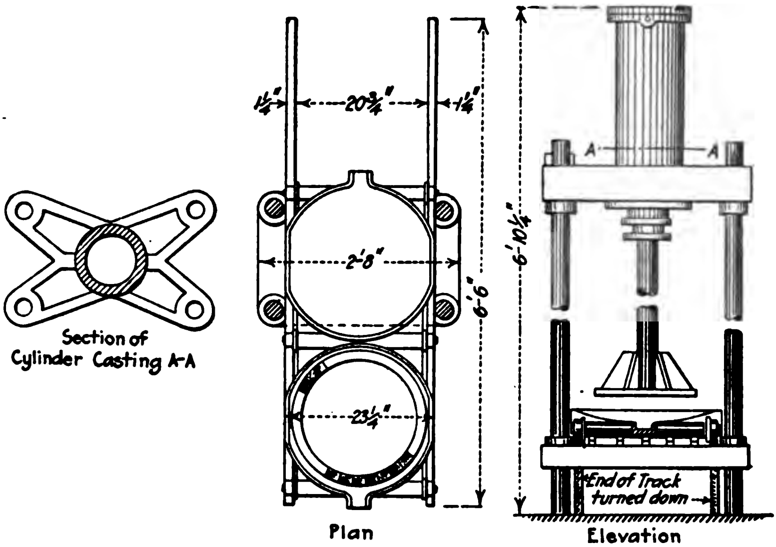


FIG. 63.—Press for screenings, Ward Street station.

connected to an engine or motor which actuates a gear wheel fastened to the shaft carrying the chain wheels. Fig. 62 shows the arrangement for providing power for two of the screens in one channel at the East Boston Station. Companion screens in the other channel have similar arrangements, and the ends of the shafts are fitted with a wheel for rope drive by which all of the screens can be operated by one motor in case of accident to the other. The cost of the apparatus has also been considerably reduced in recent designs by the substitution of standard steel shapes for much of the specially designed work at the Ward Street Station.

The screenings are burned under the boilers after being prepared by pressing out some of the water in an apparatus illustrated in Fig. 63. This consists of a steam cylinder and two tray or dish-shaped re-

ceptacles which move on a short track and may be alternately placed under the piston. For confining the screenings, a tub or cylinder without top and bottom is used. The sides of the tubs are hinged so that they may be locked, closed or thrown open for removing the pressed screenings. A tub with screenings is pushed under the cylinder and the piston head moves down into the tub. After pressing, the tub is pushed out, another one coming into place, and the pressed screenings from the first removed by throwing open the hinged sides. The pressed material is carried to the boiler house in wheelbarrows. In the early designs wooden tubs were contemplated, but experiments at Ward Street with wire gauze and other materials finally led to the use of galvanized steel perforated with small holes. These cylinders are made about 18 in. in diameter and 24 in. high.

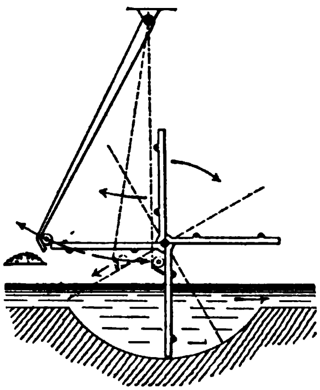


FIG. 64.—Frankfort wing rack.

Wing Racks.—One of the leading types of moving racks originated in a desire to have a rack which could be turned about an axis above the level of the sewage, thus avoiding the wear and other difficulties due to having moving pulleys and chains below the surface of the sewage. A single rack would hardly answer for such an installation, because as it moved upward out of the sewage, unless another rack had previously been inserted to the bottom of the channel, more or less suspended matter would be carried past the screening plant. It was accordingly necessary to attach to the horizontal axis about which the racks were swung at least three of these sets of bars (Fig. 64) and to construct in the bottom of the sewage channel a transverse depression through which the arms of the racks could sweep as they swung into the correct position. Such a trough is essential in order that the entire channel may be guarded by a rack at all times. If only 3 or 4 racks are used on the axis, their size must be made larger than where more are used, and the trough across the bottom of the channel must be made deeper. Four racks give the most satisfactory distribution of screening surface, in some respects, but the depth of the transverse trough must be considerably greater than is the case with 5 sets of bars, and the latter number, or even 6, is recommended by German engineers where a plant must be installed for handling considerable quantities of sewage. Where the flow is small, 3- and 4-rack installations of this sort are considered suitable.

An experimental rack of this type, constructed at Wiesbaden by Schneppendahl in 1899, operated so well that Uhlfelder adopted the principle in designing the reconstruction of the Frankfort treatment works. The apparatus has worked so satisfactorily that it has been standardized by R. Böcking & Co., of Halbergerhütte, and is sold under the name of the Frankfort clarifying rack. The 5 racks of the wing apparatus at Elberfeld are 3 meters (9.8 ft.) long and 2 meters (6.6 ft.) wide; the bars are 4 × 30 mm. (0.16 × 1.18 in.) in cross-section, and the distance between them 10 mm. (0.4 in.). The apparatus is revolved at the rate of 2½ to 5 revolutions per minute, by an electric

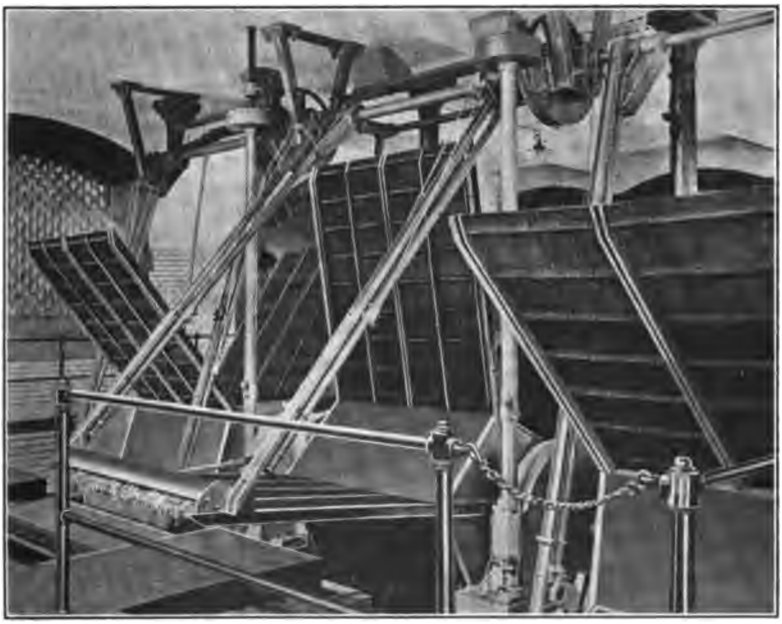


Fig. 65.—Wing screen at Frankfort, Germany.

motor requiring 2.3 h.p. The cost of operation is said to be about one-sixth that of the former manually cleaned racks. A plant of this sort at Stralsund, a place of 33,000 population, has a rack 5 meters (16.4 ft.) in diameter with 4-mm. (0.16 in.) openings between the bars, and is driven by a 4½-h.p. electric motor. The plant cost about $5200, and requires the services of one attendant. A view of the Frankfort installation is shown in Fig. 65.

The cleaning apparatus is one of the distinctive features of this type of moving rack. A pendulum arm carries a brush at its lower end, which pushes the screenings from the inner edge of each rack toward the

outer edge, finally delivering them upon a belt conveyor. The face of the brush is protected by a light rake or comb. The rack revolves in a direction contrary to the current, and hence the pressure tending to force the screenings through the openings in the rack is greater than that due to the current itself. This somewhat increases the power required to turn the rack, and if the openings in the latter are small and the sewage contains considerable quantities of suspended matter, the partial clogging of the rack may increase the amount of power considerably above that required theoretically. The longer the period a rack is submerged, the greater the quantity of screenings upon it and hence the greater the power required to move them against the current

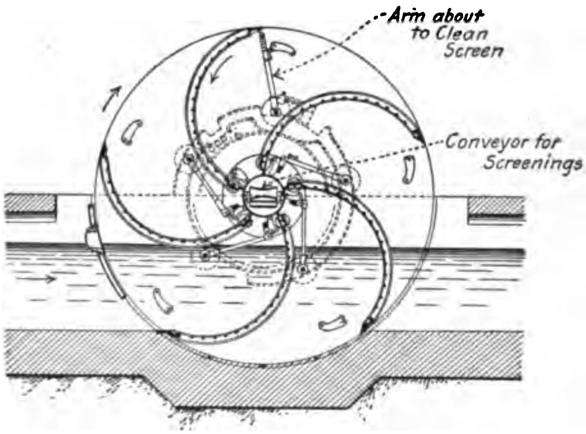


FIG. 66.—The Geiger rack.

and to raise them from the sewage. On the other hand, if the rack is moved fast, its resistance in passing through the sewage and the tendency of the current to drive the solids through the screen will both be increased.

A modification of this form of apparatus devised by Geiger has been installed at Strassburg, Bingen, Carlsruhe and several smaller places. The racks are curved in section instead of being straight (Fig. 66), and are cleaned by means of arms forming part of the revolving apparatus. These arms revolve from the outer ends of the racks toward the inner ends, where the screenings are discharged onto a belt conveyor running through the large hollow trunnion or shaft which forms the axis of rotation. It is known as the "shovel screen" and apparently does good work, but requires more power to operate it than the separator disk screens described later. One installation has been described by Allen as follows:

"The population of Gleiwitz is about 67,000, and produces a volume of sewage varying from 1,000,000 to 2,600,000 gal. per day. Two Geiger screens, 12.5 ft. in diameter and 5.9 ft. wide, are installed. The screen vanes are composed of V-shaped rods 0.12 in. apart. The quantity of material removed per year is 4700 cu. yd. or 0.19 cu. yd. per 1000 persons daily, at an annual cost of from $480 to $710. The cost, therefore, is probably about 90 cts. per 1,000,000 gal., and the cost of screenings, 12½ cts. per cubic yard. Although the average quantity of screenings is 12.9 cu. yd. per day, about 21 cu. yd. have been removed in one day, and 5 cu. yd. in 2 hours, by the two screens. The cost of each screen was $2800 and that of the entire plant $6860. Two men are required to attend the screens and pumps, one during the day and one at night. The power for operation varies from 1 to 2 h.p." (*Proc. Am. Soc. C. E.*, August, 1914, page 870.)

The results of analyses given by Allen show that the suspended solids are reduced from 2113 to 785 parts per 1,000,000, equivalent to a clarifying efficiency of 62.9 per cent. On the other hand, the solids in solution were increased about 1.9 per cent.

Link-bar Racks.—The best-known example of the link-bar rack is at Hamburg. The first plant was constructed in 1904 from the plans of Merckel and Brunotte on the right (north) bank of the Elbe; later another installation of the same type (Fig. 46) was constructed on the left (south) bank. The design was made necessary by the great range of tide at Hamburg and the elevations of the sewers. At one of these stations the sewer invert is at El. 2.5 meters (8.2 ft.); the floor of the grit chamber house had to be at least at El. 7.0 meters (23 ft.) on account of the high-water conditions in the river. In order to remove the screenings, the upper part of the rack must be at least at El. 8.3 meters (27.2 ft.). The vertical height of the rack, therefore, must be at least 5.8 meters (19 ft.). If a fixed rack were used it would be necessary to have bars at least 7.0 meters (26.2 ft.) long and the danger of their bending would be very great.

This type of rack is essentially a very broad endless link belt. The openings in the rack on the north bank are 15 mm. (0.6 in.), and those of the rack on the south bank 10 mm. (0.4 in.). The links are bars 36 cm. (14.2 in.) long, held in angle iron frames about 3 meters (9.8 ft.) long and 38 cm. (15 in.) wide. They are so attached that they can be easily taken out and replaced. Each rack consists of 46 of these frames with a total of 14,000 links. Originally the space between the frames was left open, but subsequently these spaces have been closed by rubber strips; in a later design of the frames the space between adjacent units is reduced so that the rubber is unnecessary. The original links were made of treated wood, later of hard rubber, and recently an aluminum alloy has been used exclusively. The rack moves around sheaves at the top and the bottom and can be raised so that its entire length is out of the sewage. It is cleaned by a rake or stripper having

a long row of rubber teeth passing across the entire width of the rack. As the links are quite small and, therefore, unlikely to bend, the teeth fit closely between them and clean them thoroughly.

The operation of the racks requires about 2½ h.p. and an additional 2½ h.p. is required for the cleaning apparatus. In later plants, such as those at Crefeld, Hanau and at Schoneberg near Berlin, these power requirements have been reduced. The racks of the south-side plant at Hamburg require but 1 h.p., and are operated by a Diesel engine. The apparatus is manufactured by the Maschinen Fabrik Buckau in Magdeburg.

SCREENS

Screens of perforated metal or wire mesh for clarifying sewage are practically all of the movable type. There are very few in use in the United States, as such appliances are mainly intended for fine screening, which has not found much favor for American conditions. The coarse screens or gratings are usually modifications of the coarse screens of small water works plants, consisting of perforated sheets or wire mesh in a frame capable of being raised vertically for cleaning. Neither material is usually regarded as so good for the rough service of screening sewage as racks of flat or square bars, owing to the relatively smaller percentage of open area in the perforated metal and the inferior strength of wire mesh. The difficulty of cleaning the wire mesh screens constitutes another objection to its use for screening raw sewage.

Drum Screens.—Drum screens in America are long cylinders, whereas in Germany the axis is much shorter than the diameter. One of the first, according to Frühling, was installed at Niederschöneweide, near Berlin, in 1906. This was a cylinder of brass wire cloth with 0.04-in. openings, installed primarily to intercept wool fibers. It was cleaned with jets of compressed air, which were subsequently used by Metzger, at Bromberg, whose investigations gave the drum screen its first strong recommendation for German favor. There are 4 of these screens at this place, each 8.2 ft. in diameter and run electrically at 1.2 revolutions per minute, requiring 1.2 h.p. per hour for this purpose. The screen plate has 0.08-in. perforations and is cleaned by air from a nozzle moved back and forth about 85 times per minute. From 70 to 75 cu. ft. are used hourly, requiring 0.3 kw. per hour for compression, which is considered too great. According to Allen, 1 screen averages about 1,160,000 gal. per day and removes about 9500 lb. of screenings per 1,000,000 gal. The screenings contain from 40 to 60 per cent. of moisture, and, after drying on warm plates, about 34 per cent. The entire mechanical plant cost about $7450, and the screen house and deep foundations about $15,000. A single attendant can manage the plant and the total expense for wages and power is given by Allen as $2.45

per 1,000,000 gal. If both screens were operated at full capacity this charge would be materially less.

The business of supplying drum screens in Germany has been made a specialty by Windschild, of Cossebaude near Dresden, who has developed the type shown in Fig. 67. Its advantage over the cylindrical type is the additional screening surface which comes into service when

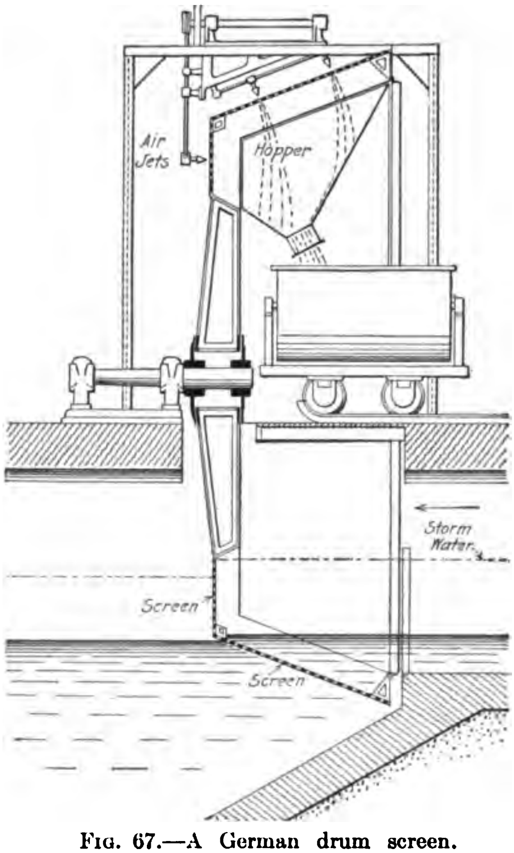


FIG. 67.—A German drum screen.

storm water flows through the sewers. There have been improvements in the air jets also, by which the air required to clean a given surface has been materially reduced from that required by the early apparatus. Some power tests of a conical screen at Mainz, 11.5 ft. in diameter, 2.73 ft. wide on the conical surface, and capable of handling 1,100,000 to 1,800,000 gal. per day, are reported by Allen (*Proc. Am. Soc. C. E.*, August, 1914, page 1875) to have indicated that from 2.0 to

2.7 h.p. were required to drive the drum at about 1 revolution in 2 minutes, and from 3.2 to 4.3 h.p. to furnish compressed air. The air used was considered needlessly large in quantity and high in pressure.

Information concerning the operation of such a screen at Trier is given by Allen. The population of 40,000 furnishes about 1,270,000 gal. of sewage daily, of which half passes off in 9 hours. The screen is 14.5 ft. in diameter, 3.94 ft. wide and rotates once in 3 minutes. The perforations of the plate are 0.1 in. in diameter and very close together, only 0.04 in. of metal remaining between the holes. Air under a pressure of about 30 lb. per square inch is used for cleaning. From 0.39 to 0.42 cu. yd. of screenings are removed daily, as a rule, although on one occasion 5.2 cu. yd. were removed in 11 hours. The screenings contain from 50 to 60 per cent. of water and are sold to market gardeners for about 14½ cts. per cubic yard. The plant cost $8600, including buildings. The plant is cared for by one laborer and a helper and the total expense for power, labor and supplies is about $1120, or about $2.41 per 1,000,000 gal.

The drum type of fine screen was first developed for sewage in the United States by the late O. M. Weand, of Reading, Pa. The designer described the first screen substantially as follows: It consisted of a rigidly constructed cylindrical steel frame 6 ft. in diameter and 12 ft. long (Fig. 68), covered with 40-mesh[1] monel metal wire cloth which was protected by a ⅝-in. mesh screen of No. 12 copper wire. This covering was put on in segments, 2½ ft. square. The machine revolved at an average speed of 8 revolutions per minute on chilled cast-iron wheels with case-hardened steel roller bearings, encased in boxes designed to be water-tight and acid-proof. The raw sewage entered the machine through a sheet-steel flume and dropped from it sideways

[1] The actual size of the openings in wire cloth is variable. The cloth used in experimental work at Chicago by Pearse had the following properties:

"By referring to a wire cloth catalogue, we find that 20-mesh wire cloth is made from 16 different diameters of wire, ranging from 0.0095 to 0.032. A 20-mesh sieve made from wire 0.0095 in. in diameter has an opening 0.0405 in., while a 20-mesh sieve made from 0.032-in. wire has an opening of 0.018 in. When we say '20-mesh,' then, it may be any screen with 20 openings to the lineal inch made of anywhere from 0.0095 to 0.032-in. wire, with a chance for variation in the sieve opening from 0.0405 to 0.018 in., a difference of 0.0225 in. so that 1 person may have 0.018 in. in mind when referring to a 20-mesh screen and another an opening 225 per cent. larger, or 0.0405 in." (G. A. Disbro, *Proc. Am. Soc. Test. Mat.*, 1913, page 1053.)

Meshes per inch	4.0	6.0	10.0	16.0	20.0	24.0	30.0	40.0
Diameter wire, inch	0.048	0.034	0.026	0.021	0.016	0.015	0.012	0.010
Net opening, inch	0.198	0.137	0.072	0.042	0.034	0.029	0.022	0.015
Open area, percentage of whole area.	65.0	64.0	54.0	42.0	46.0	42.0	41.0	28.0

on the screening cloth; the liquids passed through the openings while the solids were carried along slowly by a worm on the inside of the cylinder to the further end, where they were discharged. The meshes of the machine were kept free from clogging by oscillating jets of water, and occasionally the water was turned off and steam blown through the nozzles in order to remove the grease and slime on the cloth. The segments were also washed every 4 to 6 weeks in caustic soda to accomplish the same object.

This screen went into operation in January, 1908, and was put out of commission in August, 1912. During most of this period, it was under the management of E. Sherman Chase, chemist in charge of the sewage works, who informed the authors that the average sewage flow was 5,500,000 gal. daily, the estimated contributing population was about 40,000, the water required for cleaning was from 5000 to 10,000 gal. daily, 5 h.p. was needed for running the screen, the screenings weighed 1500 lb. per 1,000,000 gal. when wet

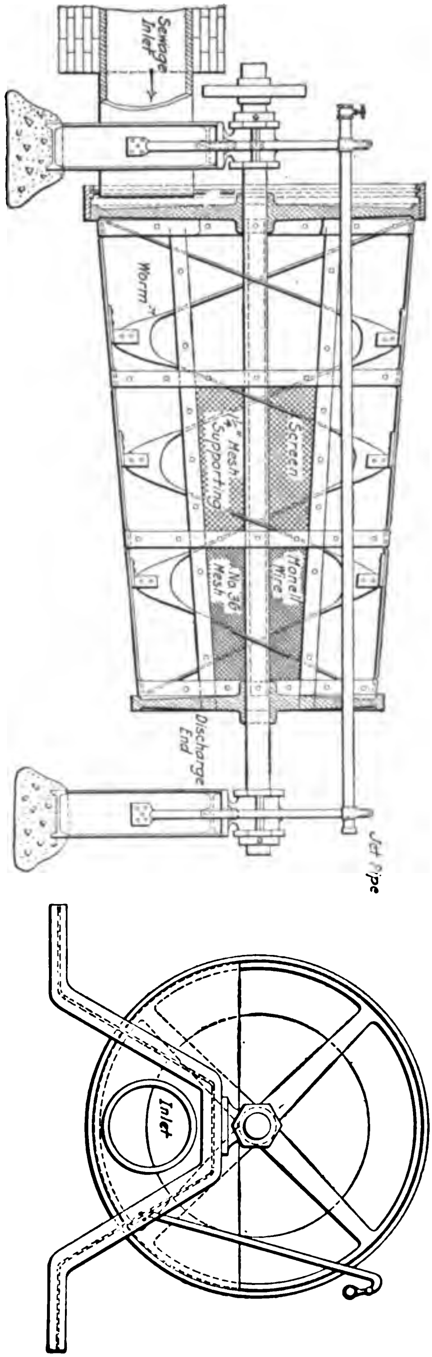


FIG. 68.—The Weand drum screen.

and 950 lb. when dried, the wet screenings contained about 90 per cent. of moisture and after they were dried in a centrifugal machine this was reduced to 75 per cent.; the screenings weighed 60 lb. per cubic foot wet and 35 lb. dry, and about 20 per cent. of the suspended matter in the sewage was removed by the screen.

The screenings were handled by 3 men in 8-hour shifts. The approximate cost of screening, excluding repairs, depreciation and other related items, was estimated by Chase at $1 per 1,000,000 gal.; the total cost, including maintenance and repairs, he estimated at $2. Each washing to remove grease took about 30 hours of extra labor. "The amount of material actually removed from the sewage by the screen was undoubtedly relatively large," Chase stated, "but the cost of its operation, maintenance and especially repairs was such that I advised the discontinuance of its operation." Another reason for abandoning the use of the segregator was that its action was largely duplicated by a settling tank which also formed part of the works for preparing the sewage for trickling filters.

A Weand segregator was placed in operation at Brockton, Mass., in 1911. It is 12 ft. long, 6 ft. in diameter and has 72 sq. ft. of screening area. It was originally covered with a 38-mesh monel metal wire cloth, which was replaced subsequently about once a year by a 30- or 32-mesh wire cloth, supported by an outer covering of coarse screening made of No. 9 copper wire with ½-in. mesh. The screen was driven by a 20-h.p. oil engine. About 20,000 gal. of water were stated to be used daily in washing the screen in 1912. In a letter to the authors in June, 1914, City Engineer B. R. Chapman made the following statements regarding this screen:

"The screen is run continuously, sewage being pumped directly from the end of the main intercepting sewer into the interior of the screen, which has a capacity of 3,000,000 gal. per day, and discharges into a 400,000-gal. receiving reservoir. When the screen was first installed, about 50 per cent. of the total suspended solids were removed, but at present only about 30 per cent. of the solids are removed, owing to the poor condition of the frame due to corrosion. The screenings amount to about 5000 lb. per day, after passing through a centrifugal dryer. The cost of operating the screen for 1913 was approximately $6000, including labor, fuel, repairs and incidentals."

The operation of the screen down to the close of 1913 is shown in Table 62, while the comments of the Sewer Commissioners in their report for that year read as follows:

"The Weand rotary screen has now been in service for over 2 years, a sufficient time to give a good idea of its efficiency, and while it has given quite satisfactory results as regards screenings, its cost of operation and

TABLE 62.—AVERAGE DAILY SCREENINGS AND SEWAGE FLOW AT BROCKTON SEWAGE PUMPING STATION

	1911		1912		1913	
	Sewage, gal.	Screenings, lb.	Sewage, gal.	Screenings, lb.	Sewage, gal.	Screenings, lb.
January	....	....	1,560,000	4,749	2,156,000	3,179
February	....	....	1,640,000	5,150	1,691,000	3,425
March	....	....	1,668,000	3,462	2,017,000	4,301
April	....	....	1,653,000	3,180	1,917,000	3,741
May	....	....	1,626,000	5,360	1,821,000	5,414
June	....	....	1,732,000	6,924	1,989,000	4,313
July	1,612,000	3,147	1,889,000	5,048	2,098,000	4,260
August	1,778,000	1,553	1,759,000	5,279	1,985,000	4,539
September	1,897,000	956	1,701,000	5,238	2,059,000	4,799
October	1,947,000	2,632	1,752,000	5,303	2,565,000	4,130
November	2,171,000	3,256	1,912,000	4,998	2,758,000	5,045
December	1,850,000	5,437	2,131,000	3,996	3,039,000	4,787
Average	1,876,000	2,830	1,752,000	4,891	2,174,000	4,327

Note.—The weights given are those of the screenings after passing through a centrifugal machine which removed an average of 19 per cent. of the water in a 3-minute run, 34 per cent. in 7 minutes, and 35 per cent. in 9 minutes.

Brockton's population was 56,878 in 1910. It has been estimated by the Water Commissioners as 60,300, 62,500 and 65,200 in November of 1911, 1912 and 1913 respectively. The quantity of sewage per capita daily during these years was, therefore, 31, 28 and 33 gal. respectively; the screening were 1510, 2790 and 1990 lb. per 1,000,000 gal. or 47, 78 and 67 lb. per 1000 population daily.

especially its maintenance in good repair have been very costly compared with newer methods of obtaining the same results. The constant liability of the screened sewage to contain paper, matches, etc., due to the sudden development of leaks in the screen itself, makes its effluent unsuitable for use on a sprinkling filter. However, the principal objections are its cost of operation and cost of renewing the screening material itself."

The screenings at this plant, as at Reading, were discharged from the trough at the rear end of the screen into burlap bags. Six of these were placed in a centrifugal machine which extracted from 19 to 35 per cent. of the water in runs of 3 to 9 minutes, during tests by A. F. Allen and Frank H. Kennedy. The maker of the machine claimed it would extract 60 per cent. in 10 minutes. The bagged screenings were reported by Allen and Kennedy to weigh 52 lb. per cubic foot after passing through the machine. At Reading these screenings were mixed with coal and burned, and at Brockton they were used for filling, being covered with ashes or soil as fast as they were spread.

The first fine screen installed at the Baltimore sewage treatment works proved unsatisfactory mechanically, and was replaced by one designed and installed by the engineers at the works. The following notes

regarding it were furnished in 1914 by Calvin W. Hendrick, Chief Engineer, Baltimore Sewerage Commission:

"It screens settled sewage, not fresh sewage.

"It is cylindrical, with one blank end, and is supported horizontally. The screening surface is 11½ ft. in diameter by 9¼ ft. long, the area of the surface effective at a given time being 175 to 200 sq. ft. The screening material is monel metal cloth, 26 meshes to the inch, supported by heavy copper wire mesh. The screen is cleaned by a series of water jets playing on it as it revolves, the jets being oscillated continually.

"The screen is capable of screening 25,000,000 to 30,000,000 gal. of settled sewage per day. No information is available (May, 1914) as to the amount of the screenings.

"The screen makes it possible to dispense with about 50 man-hours of labor daily for nozzle cleaning, and in addition keeps the sprinkling beds looking good practically all the time. Without the screen there would at times be many nozzles clogged even though the labor mentioned above was used."

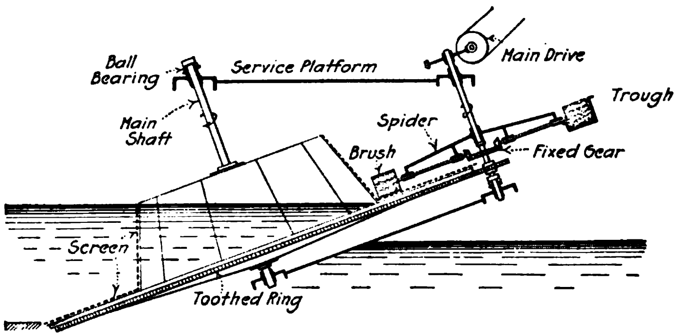


FIG. 69.—General arrangement of Riensch-Wurl screen.

Disk Screens.—It has already been mentioned that the screening area in a given channel can be materially increased by giving the racks or screens a very flat slope. This principle is followed in the design of the Riensch-Wurl separator screen, the most popular type in Germany, and controlled in the United States by The Sanitation Corporation. This was first tried at Halle in 1904. It consists of a disk of perforated metal with or without a frustum of a cone attached to its center, the whole being mounted on a shaft whose inclination from the vertical determines the tilting of the disk, as shown in Figs. 69 and 70. From 23 to 36 per cent. of the area of the disk has been given up to the openings through which the sewage passes.

The weight of the disk in screens up to 16 ft. in diameter is carried by a ball-bearing at the top of the shaft. In the larger screens the shaft is stationary and carries an annular ball-bearing support on which the

FIG. 70.—Experimental disk screen at Dresden.

FIG. 71.—Upper part of a disk screen during operation.

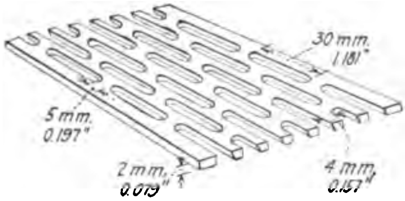


FIG. 72.—Screening plate.

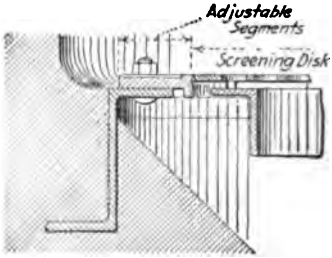


FIG. 73.—Detail of seal.

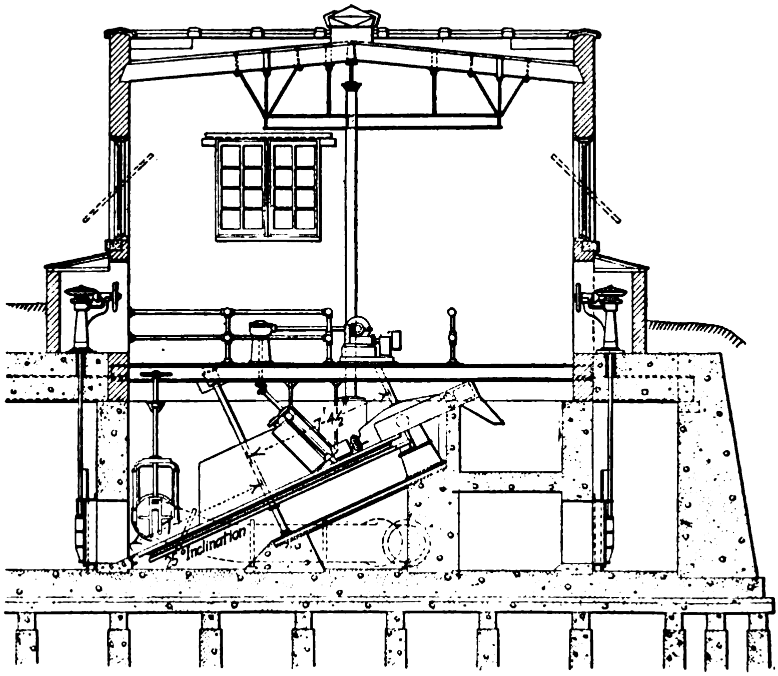


FIG. 74.—Sectional elevation of Brooklyn screen chamber. (Section taken on line BB of Fig. 75.)

frame moves. In each case the bearing is above the level of the sewage, in order that it may be kept under observation at all times. In order to provide a close seal between the edge of the disk and the side of the sewage channel, a Z-bar is embedded in the masonry with one of its flanges exactly in the plane of the top of the rim of the frame to which the disk is attached (Figs. 71 and 73). The Z-bar carries on its top a large number

of small segments which can be moved in and out so that the distance between their edges and that of the disk is not greater than the width of one of the slits of the disk. When these segments have been adjusted they can be held rigidly in position by bolts. The screenings, as they are raised above the surface of the sewage, are swept into a circular gutter by brushes on the ends of the arms of a large spider (Fig. 71). The brushes revolve and the arms also revolve, the combined motion of the several parts being such that every portion of the screening surface of the disk is passed over at least twice. This cannot be accomplished, however, with the conical screen in the center of the disk, which is

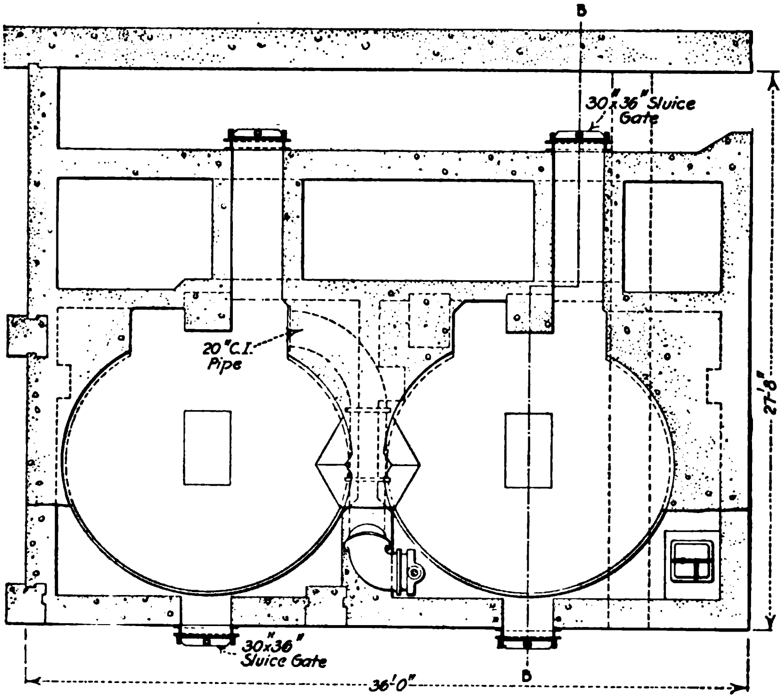


FIG. 75.—Plan of Brooklyn screen chamber showing pipe to permit series operation of screens.

cleaned by a vertical brush of conical form. Where there is much fat in the sewage it has been found desirable to blow steam or hot water over the disk once or twice a day, and the brushes are improved by the same treatment.

Over 50 of these screening installations have been made in Europe. The disks range from 1.3 to 8 meters (4.3 to 26.2 ft.) in diameter, and the width of the slots from 3/4 to 5 mm. (0.03 to 0.2 in.). The largest in-

stallations are those at Dresden and Bremen. The former is designed to intercept 100,000 cu. m. (130,800 cu. yd.) of screenings annually, and each disk is rated at 450 liters (119 gal.) per second. The population at the time the design was made was 550,000, of which half lived in buildings connected with sewers. The dry-weather flow is about 26,500,000 gal. per day and the storm-water flow rises to 110,000,000 to 170,000,000 gal. The sewage at first passes through a grit chamber, then through coarse racks with 2.6-in. openings, and finally through 4 screens each 8 meters (26¼ ft.) in diameter, with 0.08 × 1.2-in. slots. Each screen is driven by an electric motor. The speed is 1 revolution in 3 minutes, increased 50 per cent. during storms; the power required is said by Allen to be only 3.4 to 4 h.p. The channels leading to these screens are so arranged that the screened sewage from one disk can be sent for further screening through another, if so desired. The disks are set on an angle of 1:2½. The farmers take away the screenings. The Bremen plant consists of four screens 6 meters (19.7 ft.) in diameter and two, 7 meters (23 ft.) in diameter. These have slots 3 mm. (0.12 in.) wide.

An installation of Riensch-Wurl screens by E. J. Fort, Chief Engineer of the Bureau of Sewers, Borough of Brooklyn, for the experimental station of the 26th Ward sewerage system of that borough is shown in Figs. 74 and 75. This ward has a combined system serving an area of about 5000 acres, where the population increased from 13,000 in 1880 to over 200,000 in 1914, and was growing rapidly in that year. The dry-weather flow in 1914 was from 18,000,000 to 22,000,000 gal. per day. The ordinary storm flow was 300 to 500 cu. ft. per second, and the heavy storm flow was about 1000 cu. ft. per second. The amount of suspended matter varied widely at different seasons and at different hours of the day.

The two screens form a part of the permanent sewage treatment plant at this place. Each is rated at 6,000,000 gal. per day. The installation has been planned so that additions to the number of screens can be readily made. A 20-in. cast-iron pipe is provided between the screens to enable them to be operated in series, if desired, as at Dresden. It is proposed to drive the screens by a 15-h.p. steam engine. Each screen is estimated to require 4 h.p., so there will be 7 h.p. available for driving the conveying apparatus for removing screenings from the building.

The size of the apertures in the screen plates will be determined experimentally with 4 complete sets of plates. The slots in the first set will be 5/64 in. wide, in the second set 1/16 in., in the third set 3/64 in., and in the fourth 1/64 in. All slots will be 2 in. long. The bronze screen plates will be 1/8 in. thick and cross-section of the slots will be wider at the bottom than at the top. The number of these apertures is required to be

enough to permit 6,000,000 gal. of sewage to pass in 24 hours with a 12-in. maximum head on the screen. The screens are required to remove practically all particles of suspended matter with a diameter 50 per cent. greater than the width of the slot, and the specifications state that "the dissolved organic matter shall not be materially increased in passing through the screen."

Belt Screens.—Endless belt screens are used quite extensively in England, and there is a notable plant in Germany. Installations of the type shown in Fig. 76 have been furnished by John Smith & Co., Carshalton, Surrey, England, whence their name of Carshalton screens. A belt of this sort is stated by the makers to be practicable for handling quantities up to 3,600,000 gal. in 24 hours; with more than that quantity it is considered preferable to use 2 screens. One foot width of belt is considered desirable for every 460,000 U. S. gal., according to Allen. The belt

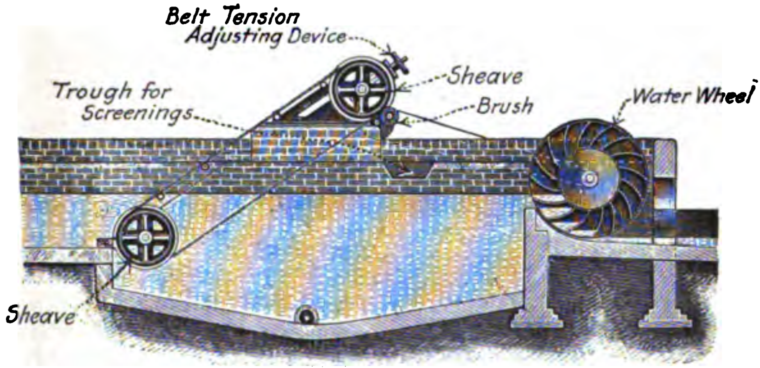


FIG. 76.—A typical English belt screen.

consists of twisted steel wires and perforated plates, the size of the holes varying from ¼ to ½ in.

A plant of this sort at Göttingen, Germany, using a belt of 1.5 mm. (0.06 in.) brazed copper wire forming square meshes measuring 10 mm. (0.4 in.) on a side, is rather unusual, because, owing to its angle of 40 deg. with the vertical, it was necessary to place small brass angles horizontally on the screen at intervals of about 1 meter (3.3 ft.) to prevent loose material from falling back into the sewage. In this installation the screen is moved at a velocity of 4 cm. (1.6 in.) per second, and the lower drum is in a cross-channel 2.3 ft. deep in the invert of the sewer in order to avoid restricting the size of the latter. This screen is cleaned not only by a cylindrical brush but also by jets of water under a pressure of about 55 lb. The screenings are washed into a tip car, one side of which is perforated to allow the water to flow away. They are composted with peat dust and sweepings to form a fertilizer.

At the Ashold works of the Birmingham, Tame & Rea District Drainage Board, Table 63, the sewage is passed through an electrically driven endless belt screen at the outlet of a grit chamber having a capacity of 780 cu. ft. According to John D. Watson, Engineer to the Board, the total suspended solids in the sewage average 210 parts per 1,000,000. The operation of the brushes cleaning the screens was found to break up large solids and force them through the openings, and one of them was abandoned when it needed repairs. The Board's chemist, F. R. O'Shaughnessy, reported on this head as follows:

"Fecal matter forms a colloidal solution when agitated with water; and the increase in the oxygen-absorbed figure together with the increase in the colloidal matter present in the sewage after passing the screen, showed conclusively the objectionable action of the screen."

TABLE 63.—COST OF SCREENING AND GRIT REMOVAL, ASHOLD WORKS, BIRMINGHAM, TAME AND REA DISTRICT DRAINAGE SYSTEM.

Year	Millions U. S. gal. sewage daily	Cubic yards of material removed annually by			Annual charge for attendance, maintenance and interest	Cost or removal of material per	
		Screens	Grit chamber	Both		Cubic yard	Million U. S. gal.
1906	2.69	276	1,616	1,892	$1,393.14	$0.73	$1.69
1907	2.82	253	1,501	1,754	989.70	0.56	1.15
1908	2.96	273	1,379	1,652	999.76	0.60	1.10
1909	3.10	262	1,501	1,763	1,403.38	0.79	1.48
1910	3.23	308	1,499	1,807	914.55	0.50	.93
1911	3.34	323	1,020	1,343	1,245.42	0.92	1.21

In July, 1913, a belt screen was put in operation at the Union Stock Yards at Chicago. This consists of removable frames attached to two parallel chain belts. Each frame holds a rectangle of 40-mesh wire cloth made of monel metal. The screen is inclined and the refuse is blown from it by compressed air. The apparatus was designed by C. A. Jennings and removes about 1000 lb. of refuse per 1,000,000 gal. of stock yards sewage. This screen is cleaned not only by a cylindrical brush but also by jets of water under a pressure of about 55 lb. The screenings are washed into a tip car, one side of which is perforated to allow the water to flow away.

AMOUNT OF SCREENINGS

The available information regarding the amount and character of screenings removed from sewage is extremely small in quantity and poor in quality, which is not surprising in view of the lack of attention paid to

the subject until recently. More data, compiled upon a uniform, comparable basis, are greatly needed regarding the operation of sewage screens. Such data should comprise an accurate description of the type of screen used, including the area exposed to the sewage, the size of screen opening and proportion of open space, the size of wire or thickness and width between openings of plate screens, the angle of inclination, the method of cleaning, the power required for operation and the cost of installation and operation. The average, minimum and maximum rates of sewage flow, the population served and a description of the industrial wastes, both as to quantity and quality, should be recorded. The volume and weight of screenings, and the proportion of water contained in them should also be given. Whether it is practicable to ascertain, even with approximate accuracy, the screen efficiency by determining the suspended solids in the sewage before and after screening is doubtful, for a very large proportion of the substances removed, especially with coarse screens, is of such a nature that it cannot be fairly sampled by practicable methods. However, such determinations are likely to lead to an under- rather than an overestimate of efficiency. In recording quantities of screenings and efficiencies, it is desirable to state the dates of tests, for screens may be more effective in cold than in warm weather because of less tendency of the solids to disintegration and solution at low temperatures, and in the autumn than at other seasons, because of leaves washed into combined sewers.

Betriebsinspektor Scheitzow of Dresden informed the authors in 1911 that the Riensch screening plant in that city was then working on a dry-weather discharge from the sewers of 90,000 to 100,000 cu. m. (23,800,000 to 26,400,000 gal.) daily, from which 16.2 to 21.6 cu. ft. of dry matter were removed per 1,000,000 gal. of sewage. The first heavy rain after a drought greatly increases the quantity of screenings; thus in the middle of September, 1911, during the first storm after 10 weeks drought, 3564 cu. ft. of refuse were removed, one of the screens running for 48 hours continuously and two for 12 hours. During 13 hours in October, 256 cu. ft. of leaves and sand were removed.

G. B. Kershaw, in his "Modern Methods of Sewage Purification," states that at 8 disposal works where he is acquainted with the operating conditions from 240 to 1920 lb. of screenings are produced per 1,000,000 imp. gal. of sewage. This is equivalent to 187 to 1493 lb. per 1,000,000 U. S. gal.

The screenings reported officially from the plant at the Manchester, England, sewage treatment works are stated in Table 64, in which the figures for U. S. measures were obtained by assuming that the British ton is 2240 lb. and the American gallon is five-sixths of the British gallon.

The volume per 1000 persons annually was estimated by the authors on the assumption that the screenings weighed 50 lb. per cubic foot.

TABLE 64.—SCREENINGS REMOVED AT MANCHESTER, ENGLAND, FROM RACKS WITH 6, 1¼ AND ½-IN. OPENINGS

Year	Quantity of sewage U. S. gallons daily	Population connected to sewers,	Screenings		
			Long tons annually	Pounds per mil. U. S. gal.	Cubic yds. per 1,000 persons annually
1902–3.....	44,620,000	567,600	3,000	413	8.8
1903–4.....	43,240,000	574,100	4,000	569	11.6
1904–5.....	36,060,000	575,300	3,238	551	9.3
1905–6.....	43,130,000	575,900	3,068	437	8.9
1906–7.....	42,600,000	576,600	3,218	464	9.3
1907–8.....	43,910,000	577,200	3,447	482	11.8
1908–9.....	44,980,000	588,600	4,059	554	11.4
1909–10....	46,540,000	603,900	4,024	530	9.7
Average....	43,135,000	560,900	3,507	500	10.4

Dr. Dunbar's investigations are summed up in his "Principles of Sewage Treatment" as indicating that from 4.7 to 9.5 cu. yd. annually per 1000 population will be removed by screening, the amount depending largely on the openings in the screens. In one case, however, having grit chambers and screening arrangements which he considered technically as near perfection as is practicable, the grit chambers and screens together removed 28.5 cu. yd.

TABLE 65.—SCREENINGS REMOVED BY RACKS WITH 1-IN. OPENINGS ON THE METROPOLITAN (BOSTON) SEWERAGE SYSTEMS

Year	Daily sewage, gal.	Screenings		
		Annually, cu. yd.	Per million gal., cu. ft.	Per 1000 persons annually, cu. yd.
North System				
1910	59,000,000[1]	2,335	2.7	4.4
1911	52,800,000	3,714	5.2	6.8
1912	55,700,000	4,069	5.4	7.3
1913	56,600,000	4,056	5.3	7.1
Average......	56,000,000	3,543	4.6	6.4
South System				
1910	39,600,000	2,312	4.3	6.4
1911	42,000,000	2,439	4.3	6.6
1912	48,200,000	2,546	3.89	6.7
1913	53,020,000	2,517	3.51	6.4
Average......	45,700,000	2,453	4.00	6.5

[1] The flow in this and previous years was overestimated.

The screenings removed from the sewage of the North and South Metropolitan Sewage Systems of Boston, where 1-in. racks are employed, are reported in Table 65, compiled from the reports of the Metropolitan Water and Sewerage Commission.

The effect of fine screening on a very weak domestic sewage (39th Street) and a very strong industrial sewage (Stock Yards) was studied

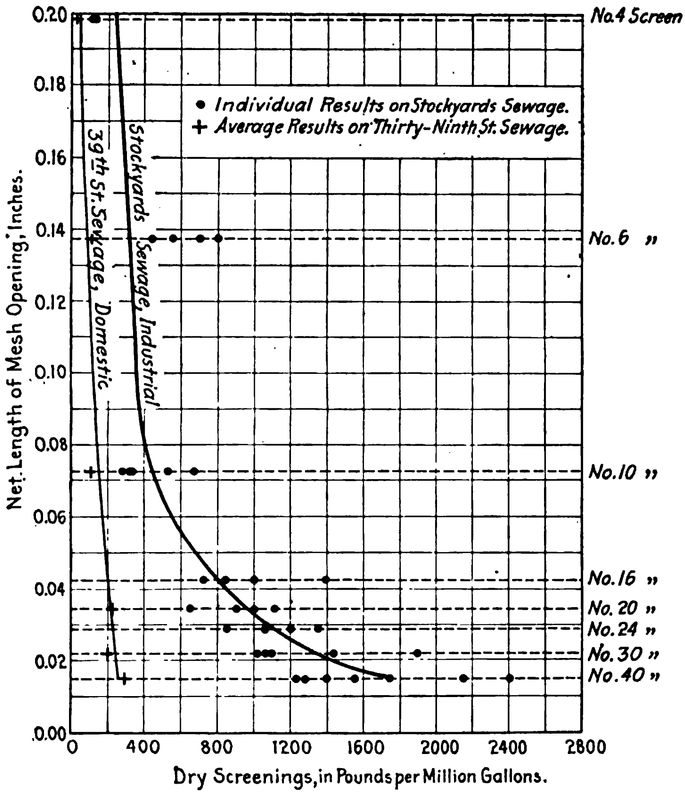


FIG. 77.—Removal of suspended matter by fine screens, Chicago. (From "Proceedings of the American Society of Civil Engineers," December, 1914.)

by Pearse at Chicago for the Sanitary District. The results are summarized in Fig. 77, from *Proceedings* of the American Society of Civil Engineers, December, 1914, page 3160. The screens were 4.2 sq. ft. in area and had openings of the sizes indicated on the diagram. At 39th Street they were under a constant head of 6 in. and at the Stock Yards under a head of 4 ft. 6 in. It will be observed that fine screening was of appre-

TABLE 66.—SIZE OF OPENINGS AND QUANTITY OF SCREENINGS.—(Continued)

Screen openings, in.	City and type	Screenings: Weight, lb. per cu. ft.	Screenings: Per 1,000,000 gal. Cu. ft.	Screenings: Per 1,000,000 gal. Lb.	Screenings: Moisture, per cent.	Screenings: Cu. yd. per 1000 persons annually	Horse-power per screen	Authority
0.5	Bradford (wing)			250				F. M. Arnolt.
0.5	Belfast (rack)	45e	5.9e	266	78.8			F. M. Arnolt.
0.5	Chesterfield (rack)	40e	6.2	250e	75.0e			F. M. Arnolt.
0.5	Eccles (rack)	40e	6.2e	250	75.0e			F. M. Arnolt.
0.5	Halifax (rack)	40e	6.0e	240	75.0			F. M. Arnolt.
0.5	Wiesbaden (wing)	60e	6.95	416e	81.0			F. M. Arnolt.
0.5	Ithaca, N. Y. (rack)			358	79.0			Schaetzle and Davis.
0.5[12]	Plainfield, N. J. (rack)	46	5.7	261	84.0			R. S. Lanphear.
0.5	Waterbury, Conn. (rack)	65	2.2	140				Schaetzle and Davis.
0.5[13]	Toronto, Ont. (rack)	38	2.8–3.7	125–171				R. C. Harris.
0.5[1]	Reading, Pa. (plate)	34	5.0	170	79.6			E. S. Chase.
0.5	Manchester, Eng. (rack)			500				Comm. rep't.
0.6	Schöneberg (link belt)		33.3	2,000	80.0	23.8		Schmidtmann; Arnolt.
0.6	Hamburg (North) (link belt)		9.2		87.0	6.6	2.5	Kenneth Allen.
0.6	Hamburg (North) (link belt)	60e	11.3	680e	83.0			F. M. Arnolt.
0.6	Leeds (rack)	40e	11.5	460	75.0e			F. M. Arnolt.
0.6	Bolton (rack)	40e	3.1	125	75.0e			F. M. Arnolt
0.6	Berlin	20	10.6	212	56.0			Emil Kuichling.
0.6	Wiesbaden (wing)		29.7			12.0		Kenneth Allen.
0.6	Charlottenburg	23	8.7	200				Emil Kuichling.
0.625[14]	Pittsfield, Mass. (rack)		1.8					A. B. Farnham.
0.71	Hannover (rack)	40e	4.5	180e	75.0e			F. M. Arnolt.
0.75	London (belt)	40e	1.7e	6.7	75.0			F. M. Arnolt.
0.75	Leicester (rack)	40e	5.1e	203	75.0e			F. M. Arnolt.
0.75	Leigh (rack)			665	65.0			F. M. Arnolt.
0.98	Clichy (Paris) (rack)	40e	21.4	865e	75.0			F. M. Arnolt.
1.0	Glasgow (cage)			20				F. M. Arnolt.
1.0	N. Met., Boston (cage)	60	5.3			7.1		Comm. rep't., 1913.
1.0	S. Met., Boston (cage)		3.51			6.4		Comm. rep't., 1913.
1.0	Providence, R. I. (rack)	80	2.4	190		29.3		J. W. Bugbee.
1.25	Newark, N. J. (rack)		1.9					E. S. Rankin.
1.50[15]	**Washington, D. C. (cage)**	54		37				A. E. Phillips.
1.625	**Atlanta, Ga. (rack)**			40				R. M. Clayton.

The notes referred to by numerals in this table are printed on pages 350 and 351.

TABLE 66.—SIZE OF OPENINGS AND QUANTITY OF SCREENINGS

Screen openings, in.	City and type	Screenings: Weight, lb. per cu. ft.	Per 1,000,000 gal.		Moisture, per cent.	Cu. yd. per 1000 persons annually	Horse-power per screen	Authority
			Cu. ft.	Lb.				
0.014[1]	Reading, Pa. (drum)	60	25.0e	1500	89.5		2.0	E. S. Chase.
0.0214[2]	Brockton, Mass. (drum)	52	38.0e	1990	71.0	(22.0)	2.5	F. H. Kennedy.
0.25	Guildford (belt)	40e	9.3e	370	75.0			F. M. Arnolt.
0.38	Burton (belt)	40e	9.5e	380	75.0			F. M. Arnolt.
0.04[3]	Wiesbaden, 1902		36.6					Emil Kuichling.
0.06[3]	Wiesbaden, 1904		52.7					Emil Kuichling.
0.06[4]	Bromberg (drum)			9200	34.3			Metzger.
0.08[4]	Bromberg (drum)			9500				Kenneth Allen.
0.08	Bromberg	65e	13.4	870e	85.0			F. M. Arnolt.
0.08	Osnabrück (drum)		98.0			50.0	9.0	Kenneth Allen.
0.08[5]	Dresden (disk)		19.0					Scheitzow.
0.08	Dresden (disk)		26.2		84.0	32.8	2.5	Kenneth Allen.
0.08[6]	Dresden (disk)		(23.0)			8.4		Frühling.
0.08[7]	Mainz (disk)			790				Kenneth Allen.
0.10	Strassburg (Geiger)		43.0		89.3	15.7	3.35	Kenneth Allen.
0.10	Trier (drum)		8.6		55.0			Kenneth Allen.
0.12	Cologne (rack)					13.1		Frühling.
0.12	Düsseldorf (rack)					19.1		Frühling.
0.12	Temesvar (Geiger)		35.1		65.0	36.5		Kenneth Allen.
0.125	Leeds	30	36.0	1080				Emil Kuichling.
0.2	Stralsund (wing)					28.9	4.5	Frühling.
0.2	Wiesbaden					26.9		Frühling.
0.25[8]	Washington, Pa. (rack)		12.0					D. C. Morrow.
0.31	Leipzig (rack)	40e	2.0	80e	75.0e			F. M. Arnolt.
0.375[9]	Columbus, O. (rack)	65	4.6	300				G. A. Johnson.
0.375	Sutton, Eng. (belt)			1344				Emil Kuichling.
0.4	Croydon (Latham)	40e	14.6e	583	75.0			F. M. Arnolt.
0.4[10]	Frankfort (wing)	40e	15.8	635e		8.8		F. M. Arnolt.
0.4	Frankfort (wing)			4000		40.9	5.0	Kenneth Allen.
0.4	Coventry, Eng.			275				Baldwin Latham.
0.4	Elberfeld (wing)	40e	21.6	865	70.0	31.0		F. M. Arnolt.
0.4[11]	Göttingen (belt)	(23)	(14.7)	(337)	83.0	9.5		Frühling.
0.4	Göttingen (belt)		10.2			10.3	2.0	Kenneth Allen.

ciable influence only with strong sewage and screens with finer mesh than about 0.08 in.

At Worcester, Mass., in connection with experimental work, sewage from a large grit chamber was taken for 18 months through a 6-in. pipe 750 ft. long to a screen chamber 2½ ft. square and 3 ft. deep, with a rack of ¼-in. bars ½ in. apart. The screenings averaged 8.8 cu. ft. per 1,000,000 gal. ranging from 4.8 to 13.1 cu. ft. The great variation was explained in part by the character of the sewage and in part by the condition of the grit chamber. When the latter was clean, the amount of screenings was often very small, and when the grit chamber was nearly full there was always an increase in the screenings.

A summary of the available information concerning the quantity of screenings obtained in experimental work and in regular service in different cities is given in Table 66.

Notes on Table 66.

Estimates by the authors or by those supplying information are marked *e*.

Emscher District data are given in Table 95.

[1] There are 2 screens at Reading, one of 40-mesh cloth and one a plate having 0.5-in. round holes. After drying, the weight of the fine screenings was 30 to 35 lb. per cubic yard. The wet screenings comprised 89.5 per cent. moisture, 2.8 per cent. mineral matter and 7.7 per cent. volatile matter. After the screenings were dried in a centrifugal machine, they contained 73 per cent. moisture, 7.4 per cent. mineral matter and 19 6 per cent. volatile matter. A screen of sheet steel with 0.5-in. round holes furnished screenings comprising 79.6 per cent. of moisture, 1.8 per cent. of mineral matter and 18.6 per cent. of volatile matter. The size of the openings of the 40-mesh wire cloth used for the fine screens was estimated by the authors and is subject to change for reasons stated in the footnote on page 334.

[2] The figures for unit weight and for moisture are for screenings dried for 3 minutes in a centrifugal machine by which about 19 per cent. of the original moisture had been extracted. The screens have about 32-mesh cloth and the size of the openings, estimated by the authors, is subject to change for reasons stated in the footnote on page 334. The screenings per 1000 population have been estimated by the authors.

[3] The screen size in 1902 is the smallest of 3 successive screens which had openings of 0.6, 0.16, and 0.04 in. respectively. The screenings recorded are for all screens. In the 1904 tests, successive screens with openings of 1.6, 0.6, 0.1 and 0.06 in. respectively were used.

[4] The published Bromberg data are confusing because several different types of screens have been tried there and the sizes of the openings differed. The weight of 0.06-in. screenings is for wet material; the moisture content given is for drained material. The screenings reported by Allen for the 0.08 screen are for dry weather; 21,500 lb. per 1,000,000 gal. are reported as the yield during periods of maximum flow.

[5] These figures were given to one of the authors during a visit to the works.

[6] These figures were obtained at an experimental plant running on sewage containing little fecal matter. The amount of screenings per 1,000,000 gal. is quoted from Kuichling.

[7] These figures refer to the permanent disk screen and not to the temporary experimental plant frequently mentioned in technical literature on screening.

[8] These are the finest racks in use in the United States (1914) of which the authors have any record. They are preceded by others with 0.625-in. openings. The daily flow averages 1,400,000 gal. The racks are cleaned hourly from 7 a.m. to 11 p.m. and every other hour during the remainder of the day. This is done by hand with rakes at an expense of about $1000 a year.

[9] There were 2 racks, one behind the other, the first with 0.5-in. openings and the second with 0.375-in. openings.

[10] The figures gathered by Kenneth Allen show only 13.4 cu. ft. per 1,000,000 gal., equal to 234 lb.

[11] The figures in parentheses are on the authority of Emil Kuichling, and the moisture content was furnished by F. M. Arnolt.

[12] These figures are for 1912 when the average flow was about 2,000,000 gal. The racks were cleaned six times between 9 a.m. and 5 p.m. and four or five times between 7 p.m. and 5 a.m.

[13] The smaller figures are for screenings from the high-level sewage and the larger for those from the low-level sewage.

[14] The racks with 0.625-in. openings are preceded by others with 0.75-in. openings.

[15] These figures refer to screenings which have been pressed, in which condition they are reduced to about 46 per cent. of their original volume and 42 per cent. of their original weight.

CHARACTER OF SCREENINGS

The character of the screenings manifestly depends greatly on whether the sewers are on the separate or combined system, on the grades and lengths of sewers, on the time it takes the sewage to reach the screens, on the cleanliness of the interior of the sewers, on the character of the population, and on any pumping to which the sewage has been exposed prior to being screened. The screenings from a 0.12-in. screen at Cologne, Frühling states, were made up of 67.0 per cent. paper, 21.9 per cent. fecal matter, 5.0 per cent. rags, 3.5 per cent. wool, hair and felt, 3.3 per cent. fruit and kitchen refuse, and the remaining 0.3 per cent. fragments of meat and offal.

The offensive character of screenings makes their prompt disposal an important matter, as is evident from a visit to most of the screening plants described in this chapter. On this head John H. Gregory has stated (*Proc. Am. Soc. C. E.*, February, 1915, page 408) that handling and removing screenings from a screening plant is just as important as handling and removing sludge from settling tanks. His observations led him to the opinion that the simplest way to avoid the nuisance was to keep the screenable materials in the sewage and handle them in settling tanks. In the same place (page 657) G. A. Soper stated that inspection of European fine screens convinced him that when they were operated without regard to the production of odors, they were the most offensive apparatus used in sewage treatment.

The composition of the screenings removed in one of the stations at Hamburg is given in Table 60, and the composition of screenings at Frankfort in Table 59.

The screenings from the Weand screen at Brockton, Mass., have been examined by A. F. Allen and F. H. Kennedy, and the analytical results in Table 67 have been furnished by them. Dried samples of the screenings furnished from 18 to 22½ per cent. of grease when extracted in ether for 1 hour.

The screenings from the experimental plant at Worcester consisted of paper, rags, garbage, fecal matter, matches, wads of hair and other sub-

TABLE 67.—MOISTURE, NITROGEN, POTASH AND PHOSPHORIC ACID IN THE CENTRIFUGED SCREENINGS FROM THE BROCKTON, MASS., SCREENS

	Average, per cent.	Maximum, per cent.	Minimum, per cent.
Moisture	71.54	76.85	63.58
Nitrogen	0.612	0.968	0.435
Potash	0.1121	0.3190	0.0742
Phosphoric acid	0.115	0.131	0.091
Ash	5.43	5.96	4.93
Calorific value	9345 B.t.u.	9429 B.t.u.	9221 B.t.u.

stances largely organic and readily digested by septic action. An analysis of the screenings is given in Table 68.

TABLE 68.—ANALYSIS OF SCREENINGS CAUGHT BY ½-IN. RACK, WORCESTER, MASS., 1911

Wet		Dry	
Weight per cu. ft., lb.	50	Organic matter, per cent.	77.6
Moisture, per cent	61	Mineral matter, per cent.	22.4
Dry solids, per cent	39	Fats, per cent	1.8

RECENT SCREENING TENDENCIES

Coarse racks for removing the grosser floating matter in sewage will continue to be used extensively. It seems probable that the day of elaborate fine screening has passed, in view of the results attained with improved sedimentation tanks. If the matter removed by fine screening can be successfully taken out in settling basins, the cost will usually be less.

In his "Modern Methods of Sewage Purification," G. Bertram Kershaw states that fine screening, as a preliminary to subsequent treatment, will be required only where it is proposed to dispense entirely with settlement before filtration and to send the screened sewage directly to the filter. Wisner reported in 1914 to the trustees of the Chicago Sanitary District that there was little or no improvement in the stability of the local sewage caused by screening through a 40-mesh screen.

On the other hand, George W. Fuller states in his "Sewage Disposal" that screens have a far greater field of usefulness than is generally recognized at present in America. He believes that there is much room for improvement upon present devices for fine screening, and he points out that their utility and standing for the removal of suspended organic

matters, in comparison with other devices, will depend upon future data from recent designs and later improvements. In a report to the Metropolitan Sewerage Commission of New York, he stated (Oct. 15, 1913) that he preferred fine screens to settling tanks only where it was desirable or necessary to remove only relatively large sewage matters in suspension. Where settling solids would form deposits in the water-courses if screening alone were adopted, he preferred to install settling tanks rather than fine screens.

The experience with fine screening at Washington, Pa., is summed up in a letter to the authors by the borough engineer, D. C. Morrow, as follows:

"Since the sewage must be pumped, it is necessary to have it well screened to avoid clogging the centrifugal pumps. The removal of a large percentage of suspended matter by screening lengthens the time that the sedimentation tanks can be operated without cleaning, and at the same time reduces the quantity of sludge to be dried on the sludge bed. However, I believe that in a plant where it is not necessary to pump the sewage, having adequate tank capacity where the sludge can be thoroughly digested before being discharged and with a well-constructed sludge bed, fine screening will be quite unnecessary."

In the *Proceedings* of the American Society of Civil Engineers for December, 1914, page 3152, Dr. Hering stated that he considered coarse screens sufficient for any sewage treatment when the sewage goes into tanks where it is supposed to be detained. He believed that light matter can be removed much more thoroughly and cheaply from a tank than from a screen. Fine screens without tanks can prepare sewage for discharge into some rivers, for intermittent filtration and for trickling filters, he stated. It was possible, also, that they might be used to screen very fresh sewage before it entered a long trunk sewer, where the coarse matter would be churned into bits and passed into colloidal solution. In the same discussion Pearse stated (page 3161) that numerous tests with a 30-mesh Weand drum screen, on sewage with a biological oxygen consumption of 1100 parts per 1,000,000, showed an improvement of only 6 per cent., with a removal of 8 per cent. of suspended matter from sewage containing about 487 parts per 1,000,000 of suspended matter. Settling the screened sewage removed 283 parts per 1,000,000 of suspended matter, making a total removal of 66 per cent. on the crude sewage. There was a reduction of 42 per cent. in oxygen demand in a number of tests. This was considered proof that with such sewage fine screening was not comparable with sedimentation.

Schmeitzner's investigations led him to the conclusion that fine screens are objectionable if they reduce the size of the suspended matter and increase the time of its subsidence (page 49, Kimberly's translation of his

"Clarification of Sewage"). He did not expect any reduction in the extent of sedimentation basins needed in treatment, because it is their office to remove the small matter which passed through a fine screen, and the fine screen merely reduces the quantity of sludge which is deposited in the basins. He could find no proof that it was cheaper to clean screens than to remove sludge from basins, but rather that the proper design of settling basins and sludge drains makes it possible to remove sludge automatically at low cost. These remarks he applied also to fine screens in the inlets of septic tanks, but he considered screens with 0.06 to 0.12-in. openings practicable in front of the outlets from such tanks, if they were detailed and operated so that they caused no material damming of the sewage and corresponding fluctuations in the rate of flow through the tanks.

Fine screening was carried on experimentally at Leeds, England, for several years, the sewage being of an unusual character on account of the large amount of wool fiber in it. In a report made in 1905 by Col. T. W. Harding and W. H. Harrison, the investigation was summarized as follows:

"A variety of experiments was carried out with screening materials, and screens of about 30 (meshes) per in. were found to be necessary. It was very difficult to keep these clear, and a series of screens, one finer than the other, was used in succession for a long time. In passing sewage, even after screening through a 1/16-in. screen, through a fine sieve of 30 (meshes) per inch a thin and almost impervious layer about 1/32 in. thick was formed in a few minutes, which made such screening impracticable. After a while, however, we devised a plan of inclined screens, the rush of the sewage washing forward the matter screened off, and ultimately we were able to dispense with the series of screens and use only the finest screen, 30 per in. This fine screen alone followed the coarse 1-in. grating at the entrance of the works, which kept back sticks, cabbage leaves, large pieces of paper and generally the coarsest solids. The fine screen kept back fiber, small pieces of paper, matches, tea leaves, and such like, together with the small particles mixed with them.

"At first it seemed as if a considerable proportion of the suspended matter in the sewage was kept back by the fine screen, but although the accumulation seemed bulky, it was found on drying to represent less than 10 per cent. of the suspended matters in the sewage. They, however, were of a kind to be very slowly reduced by bacterial action, and were just those which tended to form a felted mat on the surface of the filter, and which it was important to keep back. Fine screening, therefore, although valuable, still leaves the bulk of the suspended matter in the sewage, but in a finely divided form, which readily works its way down through a percolating filter, becoming reduced and oxidized on the way, coming out as fine brown particles in the nature of humus. Where sewage is to be subjected to settlement before filtration, there seems to be no object in having fine screening, for the matters which would be screened out would be much less expensively

kept back by settlement, together with the finer matters, so forming the sludge to be dealt with by pressing or otherwise."

Frühling's comments on the German experience with sewage screens reads as follows:

"Screening plants have removed, on an average, about 10 per cent. of the suspended matter in sewage. This apparently small operating result leads to a question whether the endeavor of recent years to improve the design and operation of screening apparatus and thereby widen their previously limited field of service has been economically justified. This question must be answered affirmatively where the character of the receiver of the sewage is such that fine screening is the only treatment needed to prepare the sewage for its discharge into the stream or lake, and its adoption makes the construction of treatment works unnecessary, at least for some years. In such cases, hand cleaning is entirely impracticable and would also be too expensive. The same is true for most screening plants guarding pumping machinery, whether their purpose is to remove coarse material from the sewage permanently or merely to transfer it to machines which will reduce it to pieces of such size that they may be returned to the sewage without threatening to interrupt the pumping. If the racks are part of extensive treatment works, coarse screening is sufficient; with mechanical operation, however, the first cost and operating charges are increased but little if the plant is designed to take out a part of the fine suspended matter, which will have less water if taken out as screenings than when removed as sludge from basins."

Screening Experience at Worcester, Mass.—In the first (1890) chemical precipitation plant, racks of ¼ × 1½-in. bars and openings of about ¾ in. were installed near the mouth of the outfall sewer, no grit chambers being provided then. One laborer was engaged in cleaning the racks with a rake during a large portion of the day in dry weather, and in wet weather, when large quantities of litter were carried by the sewage, the work was continuous throughout the 24 hours. On account of the expense thus involved, the racks were removed and the sewage was allowed to flow directly into the settling basins where the matter which had previously been removed by the racks was allowed to settle with the sludge.

At this time the sludge was pumped from the settling basins by means of a centrifugal pump, and it was found necessary to place a rack in front of this pump and delegate a laborer to the cleaning of this rack during the period of pumping, which did not average over 3 hours per day. At a later date the centrifugal pump was replaced by a Shone ejector, provided with inlet and outlet pipes 12 in. in diameter. As this machine was capable of pumping very coarse material, it was found unnecessary to provide racks. Thus by practical experiment the entire cost of screening was substantially eliminated.

For a number of years a large portion of the sludge has been partially

dried by means of filter presses. On account of the danger of clogging the valves in the pumps and the center passages in the filter presses, it has been found necessary to screen the sludge.[1] The racks, however, require the services of 1 man for cleaning only at times when the sludge is being pumped to the filter presses, generally not in excess of 8 hours per day.

[1] Kershaw states that in Birmingham it has been found more desirable to screen sludge than sewage.

CHAPTER X

SEDIMENTATION, STRAINING AND AERATION

The general purpose of sedimentation and the theory of the process were outlined on page 207. In addition to what was there said, attention is called to the advantage of sedimentation in connection with the disinfection of sewage to prevent bacterial contamination of shellfish beds and bathing beaches. The success of such treatment appears to require a short period of contact, and there is evidence tending to show that disinfection can be more effectively accomplished when the chemicals are applied to clarified rather than raw sewage. In such instances sedimentation may prove a valuable part of the treatment.

The quantity of suspended matter that will settle in basins during the detention period varies greatly with different sewages, and is largely influenced by the quantity and character of the industrial wastes and the amount of street washings reaching the sewers. The quantity to be expected can be estimated from the analytical records in Chapter V. Whenever precipitates are formed by chemical reactions going on in the sewage, the efficiency of the sedimentation process will be greater than when there is little or no such chemical action.

The relative proportions of mineral and organic suspended matter in the sewage may have some influence on the rate of sedimentation and on the quality of the resulting sludge. Results at Columbus indicated to Geo. A. Johnson that the rates of subsidence of the suspended matters in sewages of the same general character varied inversely as the percentage of organic matter (Table 69). His deductions were drawn from a comparison of the sewages of the same general strength on successive days. If a comparison is made between the organic matter and percent-

TABLE 69.—SEDIMENTATION OF STRONG, MEDIUM AND WEAK SEWAGE

(Report on Sewage Purification, Columbus, Geo. A. Johnson, p. 105)

Date, July, 1905	Percentage of suspended matter which is organic			Percentage of total suspended matter removed		
	Strong	Medium	Weak	Strong	Medium	Weak
11/12.......	54	50	35	61	52	35
12/13.......	51	46	47	70	60	19
13/14.......	39	54	50	75	58	17

age of removal in the three classes of sewage on the same day, the effect of sedimentation is seen to be directly proportional to the amount of organic matter present.

Proportion of Solids Capable of Settling in Given Time.—Steuernagel found at Cologne that 79.5 per cent. of the organic suspended matter would settle in 12 hours (Mit. Kön. Prufungsanstalt, vol. iv). Johnson found at Columbus, with a tank 200 ft. long, 8 ft. wide and 7 ft. deep, that the precipitation at a velocity of .4 mm. per second, or 47.2 ft. per hour, was most rapid in the first 40 ft. of the tank. This is shown in Table 70, from which it will be seen that while approximately 50 per cent. was removed in 1.7 hours, only 10 per cent. more was removed in an additional 2.5 hours.

TABLE 70.—PRECIPITATION WITH A VELOCITY OF FLOW OF 47 FT. PER HOUR

(Report on Sewage Purification, Columbus, 1905, George A. Johnson, page 102)

Length of travel, feet.	Period of precipitation, hours	Suspended matter removed, per cent.
40	0.8	35
80	1.7	50
120	2.5	55
160	3.3	57
200	4.2	60

In Steuernagel's sedimentation experiments at Cologne, sewage was studied by allowing samples to remain at rest in tanks 8.2 ft. high and 16 in. square in section. From time to time 7 samples were drawn from a point 6.6 ft. below the surface and the amount of suspended matter determined. The results were checked by allowing sewage to flow at different velocities through a basin having an average depth of 6.6 ft. The results are given in Table 71, and the results of other tests are given in Chapter V, page 163.

TABLE 71.—SEDIMENTATION IN TANKS AND BASINS, COLOGNE

(Entwässerung der Städte, Frühling, page 527)

Basin experiments				Tank experiments	
Rate of flow		Detention period, minutes	Suspended matter removed, per cent.	Sedimentation period, minutes	Suspended matter removed, per cent.
Millimeters per second	Feet per hour				
4	47	187.5	72.3	187.5	70.1
20	236	37.5	69.1	37.5	64.1
40	472	18.8	58.9	18.8	57.4

Fill-and-draw and Continuous-flow Operation.—The two methods of operating sedimentation basins are explained on page 209. Many plants have been operated for a time according to the fill-and-draw method but in nearly all cases it has finally been found that continuous flow is the more practical method of operation. Nearly all of the suspended matter which will settle out in a reasonable length of time when the sewage is held quiescent will also settle out when the sewage is allowed to flow very slowly through the tank. Furthermore, it is practically impossible to hold sewage absolutely quiescent in large tanks because of wind action, oscillations of the mass of sewage in the tanks, and currents set up by differences of temperature of the sewage in different parts of the tanks. There is also generally considerable delay due to cleaning the tanks between emptying and filling, especially where they are of the shallow, horizontal-flow type, and if the sludge is not removed each time before refilling it is mixed with the incoming sewage and tends to hasten decomposition, which may be objectionable because of the odors produced about the plant.

In the continuous-flow method, the sewage may leave the tank in a comparatively short time without having taken up dissolved matter present in the sludge and in some cases may contain some dissolved oxygen, an assurance that exceedingly offensive odors are not being given off. The sludge, being undisturbed, may remain for several days or, at times, even for two or three weeks before removal becomes necessary. In this way the sludge becomes consolidated and the volume removed is much smaller than when it is necessary to clean the tank between fillings in the fill-and-draw method.

Theory of Sedimentation.—Allen Hazen has investigated the theory of sedimentation from a mathematical standpoint, and with special reference to water purification (*Trans. Am. Soc. C. E.*, vol. liii, 1904, page 46). The sedimentation of sewage is, however, greatly complicated by changes of composition constantly taking place, the evolution of gas, temperature, and other conditions. The paper is a long one and only a few of its sections are mentioned in the following abstract:

Velocity at which Particles of Sediment Settle through Still Water.—The larger particles settle rapidly, the smaller ones very slowly. With very small particles the viscosity of water controls, and the velocity of settlement, or the hydraulic value, varies as the square of the diameter. With large particles friction controls, and the velocity or hydraulic value[1] varies as the square root of the diameter. There is a transition space between. This space covers particles from 0.1 to 1.0 mm. in diameter, or ordinary sand, and also extends somewhat beyond these limits.

The hydraulic values of particles within these limits have been determined by noting the time required for settlement for a determined distance through

[1] The hydraulic value of a particle of suspended matter is the velocity in millimeters per second with which it settles in still water.

water in a glass vessel. Particles of different sizes were obtained by the methods used in the mechanical analysis of sand. The specific gravity of the particles is about 2.65. The grains are irregular, and the diameters are taken as the diameters of spheres of equal volume.

TABLE 72.—VELOCITIES AT WHICH PARTICLES OF SEDIMENT FALL IN STILL WATER

Diameter of particles, in mm.	Hydraulic value, in mm. per second 10°C. = 50°F.	Remarks
1.00	100.0	Experiments by Hazen.
0.80	83.0	Experiments by Hazen.
0.60	63.0	Experiments by Hazen.
0.50	53.0	Experiments by Hazen.
0.40	42.0	Experiments by Hazen.
0.30	32.0	Experiments by Hazen.
0.20	21.0	Experiments by Hazen.
0.15	15.0	Experiments by Hazen.
0.10	8.0	Experiments by Hazen.
0.08	6.0	Interpolated from connecting curve.
0.06	3.8	Interpolated from connecting curve.
0.05	2.9	Interpolated from connecting curve.
0.04	2.1	Interpolated from connecting curve.
0.03	1.3	Interpolated from connecting curve.
0.02	0.62	Wiley's formula.
0.015	0.35	Wiley's formula.
0.010	0.154	Wiley's formula.
0.008	0.098	Wiley's formula.
0.006	0.055	Wiley's formula.
0.005	0.0385	Wiley's formula.
0.004	0.0247	Wiley's formula.
0.003	0.0138	Wiley's formula.
0.002	0.0062	Wiley's formula.
0.0015	0.0035	Wiley's formula.
0.001	0.00154	Wiley's formula.
0.0001	0.0000154	Wiley's formula.

Note.—These values are not given as being precise, but they are believed to be sufficiently accurate for the purpose of this discussion.

For particles less than 0.025 mm. in diameter, the formula given by Dr. W. H. Wiley ("Agricultural Analysis," page 212) is used, namely, $d = 0.0255v^2$, the diameter being in millimeters and the velocity in millimeters per second. The hydraulic values of particles from 0.025 to 0.1 mm. in

diameter have been obtained by drawing a curve between the lines representing the higher and lower values. Some of these values are given in Table 72.

On the Effect of Temperature.—The figures in Table 72 are for a temperature of 10°C., or 50°F., which is about the annual average temperature of the water in the northern part of the United States. The finer particles settle more rapidly as the water becomes warmer, but with the coarser ones temperature makes less difference. For the finest particles the rate of settling at different temperatures varies as $(t + 10) \div 60$, t being the temperature on the Fahrenheit scale. The relative hydraulic values of the same particles at different temperatures are as follows:

Temperature, Fahrenheit	Relative hydraulic value	Temperature, Fahrenheit	Relative hydraulic value
32	70	56	110
38	80	62	120
44	90	68	130
50	100	74	140

At a summer temperature of 74°F., a particle of sediment will settle twice as fast as at the freezing point. In other words, a given sedimentation basin will do twice as much work in summer as in winter. Experience indicates the truth of this deduction.

Résumé.—The fundamental proposition, in clearing water by sedimentation, seems to be that every particle of sediment moves downward through the water at a velocity depending upon its size and weight and upon the viscosity of the water. Particles of sediment are generally so far apart that they do not influence each other;[1] and, while there is no doubt that they do sometimes collect in groups and thus change the conditions, it seems to be generally true that each particle will settle as if no other particles were present.

If the water in a basin were absolutely quiet there would be a regular sequence of clearing beginning at the top. The coarsest particles would go down fastest, but at any given point there would be a gradual clearing, and this clearing would take place most rapidly at the top, and, after longer intervals, at lower points in the basin.

James A. Seddon started out with this theory in a paper in *Jour. Assoc. Eng. Socs.*, 1889, page 447, but found it to be not in accordance with the facts. His observation showed that while the amount of sediment in the water at the top was a little less than in the water at the bottom, the distribution was nearly equal throughout the mass, a condition of affairs inconsistent with the theory. He accounted for this distribution of sediment by the constant mixing of the water from top to bottom, and to the sustaining power of vortex motions in the water. These motions he thought arose from the

[1] Hasen was dealing with the natural sedimentation of water. When coagulant is used this statement will not apply.

internal motion of the water at the time of entrance, and from wind, and from temperature changes.

Hazen has taken Seddon's development of the case as his starting point, and has carried the discussion further. He believes that while the internal motions keep the water mixed, and with nearly the same density of sediment from top to bottom, the tendency of the particles of sediment to settle is, nevertheless, an unbalanced force always acting to take the particles to the bottom, and the number of particles that hit the bottom in a given time is proportional, first, to the velocity at which the individual particles settle, and, second, to the density of sediment in the water immediately above the bottom.

With these fundamental relations in mind, it is easy to compute and to express by simple formulas the proportions of particles of sediment of a given hydraulic value which will hit the bottom under given conditions and which, therefore, presumably, will be removed.

The fundamental propositions may be very concisely expressed. They are: first, that the results obtained are dependent upon the area of bottom surface exposed to receive sediment, and that they are entirely independent of the depth of basin; and, second, that the best results are obtained when the basins are arranged so that the incoming water containing the maximum quantity of sediment is kept from mixing with water which is partially clarified. In other words, the best results are obtained where any given lot of water goes through the basin with the least mixing with the water which entered before it, and with the water which enters after it. This is practically accomplished by dividing the basins into consecutive compartments by baffles or otherwise.

Thus far, the discussion is easy and apparently certain. The next step is a more difficult one. It relates to bottom velocities, and has to do with the question whether these velocities are such as to allow the particles to remain on the bottom when they get there, or whether they will be taken up again and be kept in motion with the body of the water. This is a point upon which further experimental data are needed. The problem of securing such data seems to be difficult. The observations must be made at the bottom of a layer of liquid of considerable thickness, where the conditions of observation are not favorable. The observations, further, must be made on very low velocities and on particles so small as to be practically microscopic.

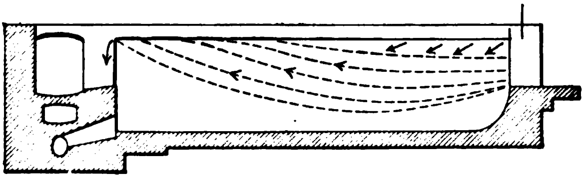
Whatever view may be taken of the second part of the problem, or whatever researches upon it may show, the arrangements of basins most favorable to taking particles to the bottom should stand.

Currents in Basins.—Besides the direct effect of the temperature on the viscosity of the water and consequent hydraulic value of small particles, changes of temperature produce disturbances interfering with the calculated velocities, which are discussed in Seddon's paper, already referred to.

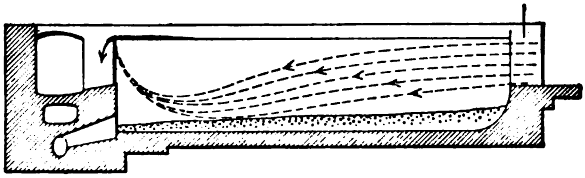
Dr. Dunbar says, in his "Principles of Sewage Treatment:"

"The direction of the sewage currents at various depths in the tanks has been investigated by Bock and Schwarz. They employed small glass bottles,

and sank these to depths varying from 1 to 6 ft. Their results showed that the sewage moved sometimes upward, sometimes downward, and sometimes toward the sides of the tanks, with a velocity two to three times as great as the calculated average velocity through the tank. These sources of error were later demonstrated in a very clear manner by Schmidt at Oppeln. By addition of a coloring matter (uranin) he showed that at the cooler periods of the year the warm sewage flowed on the top of the cooler contents of the tanks. Variations in temperature cause variations in the flow of the sewage, as depicted in Fig. 78. The dotted lines show the direction taken by the entering sewage, according as it is warmer or colder than the contents of the tank" (page 71).



Flow on Cold Days.



Flow on Warm Days.

Fig. 78.—Effect of temperature on currents in basins. (From Dunbar's "Principles of Sewage Treatment.")

Johnson states in his "Report on Sewage Purification, Columbus," (page 92) that stratification is a factor of slight importance.

Density of Sludge Deposited at High and Low Velocities.—An experiment by Steuernagel at Cologne showed that the sludge from 1000

Table 73.—Water in Sludge Deposited at High and Low Velocities

Velocity, ft. per hour	Sludge, gal. per 1000 gal. of sewage	Analysis of sludge		Relative weights of dry residue (2) × (4)
		Moisture, per cent.	Dry residue, per cent.	
(1)	(2)	(3)	(4)	(5)
47	4.040	95.57	4.43	17.9
236	2.474	92.87	7.13	17.64
472	1.838	91.34	8.66	15.91

gal. of sewage, moving at different velocities, contained the amounts of water given in Table 73. In these cases it will be seen that the amount of dried residue collected at the highest rate was almost equal to that at the lowest rate although in the former case the velocity was ten times as great. Furthermore, the quantity of wet sludge to be handled at the higher rate was less than half that at the low velocity.

Results just opposite to these have been obtained in experiments at low velocities at Worcester, Mass. (Table 74). Here the percentage of moisture in the sludge increased as the velocity of flow was increased. It is possible that these discordant results are to be explained by the different characters of the sewages. Logically, it would seem that at high velocities, when the sand and dense material are deposited while the fine light material which increases the volume of sediment at the expense of density is carried along, the percentage of moisture should be relatively low. At Worcester, however, even at high velocities, large quantities of fibrous and flocculent matter are deposited and when the velocity is lessened the fine material previously carried over is deposited and fills the interstices in the coarse material, with consequent increase in density without much increase in volume. All these experiments were conducted at velocities far below the lowest at Cologne.

HORIZONTAL-FLOW SEDIMENTATION TANKS

The results of experiments by Steuernagel at Cologne and Bock and Schwarz at Hannover were valuable in showing that the reduction of velocity below certain points did not increase the efficiency at a corresponding rate. In many of the older plants, before the futility of attempting to remove the very fine suspended matter by long sedimentation was recognized, the length of storage was very great and the consequent velocity low. Some of the English and older American plants provide for a detention period of from 15 to 20 hours. More recently the usual practice has been to provide from 2 to 6 hours' detention. Steuernagel's Cologne experiments gave the percentages of sludge settling in a sump immediately in front of the inlet to a long sedimentation basin, and on the floor of the basin beyond the sump, which are shown in Fig. 79.

At Worcester, Mass., tanks operated in 1902–1903 at various rates gave results shown in Table 74. The tanks were 40 ft. wide, 166.6 ft. long and approximately 7 ft. deep, each having a capacity of 350,000 gal. The lower part of each tank was divided into compartments by sludge dams from 2 to 3 ft. high, extending across the tank. Scum boards or baffles extending 18 in. below the surface of the sewage were placed across the tanks directly over the sludge dams. The sewage entered the tanks through apertures 18 in. apart in troughs extending across one end of the tanks. The effluent passed out by falling over a weir at the

other end extending entirely across the basins. In the table the results are based on the assumption that the bottom 2 ft. of the tanks are filled with sludge.

Table 75 gives a summary of other results of the experiments at Worcester in 1902–1903. In this table the period of detention and rate of flow are figured on the full tank capacity.

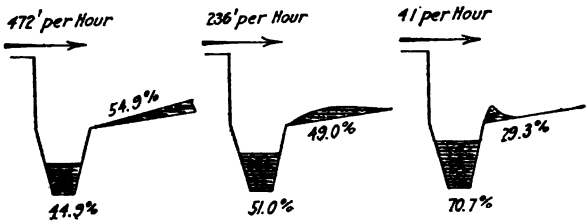


FIG. 79.—Distribution of sludge in Cologue experiments. (From Dunbar's "Principles of Sewage Treatment.")

TABLE 74.—RESULTS OF EXPERIMENTS UPON SEDIMENTATION AT WORCESTER, MASS., 1902–1903

Test period	Rate, million gal. daily	Detention period, hours	Horis. vel. ft. per hour	Sewage (parts per 1,000,000)		Effluent (parts per 1,000,000)		Suspended solids removed, per cent.	Suspended alb. ammonia removed, per cent.
				Total suspended solids	Suspended album. ammonia	Total suspended solids	Suspended album. ammonia		
1	0.3	20.8	8.0	179	3.46	110	2.41	38.5	30.3
2	0.3	20.8	8.0	300	6.10	183	2.66	39.0	56.4
2a	0.3	20.8	8.0	300	6.10	200	3.22	31.7	47.2
3	½ at 0.3	20.8	8.0	319	6.55	143	2.36	55.1	63.9
	½ at 0.4	15.0	11.1						
4	0.4	15.0	11.1	339	6.02	170	3.27	49.8	45.7
5	0.5	12.0	13.9	361	6.56	213	3.20	41.0	51.2
5a	0.5	12.0	13.9	333	5.15	156	1.51	53.2	70.7
6	0.6	10.0	17.0	364	6.97	160	3.33	56.0	52.2
7	0.75	8.0	20.8	246	3.33	83	0.60	66.3	82.0
8	0.75	8.0	20.8	284	3.33	143	1.67	49.6	49.8
9	0.75	8.0	20.8	205	2.73	101	1.03	50.7	62.3
10	0.75	8.0	20.8	291	5.03	122	2.04	58.1	59.4
11	0.75	8.0	20.8	283	4.74	125	1.81	55.8	61.8
12	1.0	6.0	27.8	333	5.13	194	2.63	41.7	48.9
13	1.0	6.0	27.8	335	6.40	163	4.31	51.3	82.6

Note.—Apparent discrepancies in the relation of velocity to reduction of suspended matter in Tables 74 and 75 were attributed largely to different detention periods and changes in the character of the sewage.

A second set of experiments with much higher velocities was made at Worcester during 2 years beginning September, 1908. Two tanks

TABLE 75.—SUMMARY OF RESULTS OF EXPERIMENTS ON SEDIMENTATION AT WORCESTER, MASS., 1902–1903

Nominal daily rate, gal.	300,000	500,000	750,000	1,000,000
Period of detention,[1] hours.	28	16.8	11.2	8.4
Horizon. veloc.,[1] ft. per hr.	5.95	9.9	14.9	19.8
Effluent:				
Total susp. solids, lb. per 1,000,000 gal.	1,390	1,528	960	1,480
Total susp. solids, parts per 1,000,000.	166	183	114	177
Sludge:				
Gal. per 1,000,000 gal. sewage:	3,610	3,282	1,671	1,614
Lb. per 1,000,000 gal. sewage.	1,900	1,480	780	480
Percentage of moisture.	93.8–93.6	95.3–93.4	96.9–92.6	96.6–96.1

[1] The period of detention is longer and the horizontal velocity of flow slower in this table than in Table 74, because of different assumptions regarding the vertical cross-section of the flowing sewage. The assumptions of less detention and higher velocity are probably more nearly in accord with the actual conditions.

TABLE 76.—RESULTS OF EXPERIMENTS UPON SEDIMENTATION AT WORCESTER, MASS., 1909

(Estimated velocity of flow, 333 ft. per hour)

Date	Sewage (parts per 1,000,000)		Effluent (parts per 1,000,000)		Suspended solids removed, per cent.	Sus. alb. ammonia removed, per cent.
	Total suspended solids	Suspended albuminoid ammonia	Total suspended solids	Suspended albuminoid ammonia		
January	342	7.31	252	5.51	26.3	24.6
February	296	5.87	210	3.60	29.0	38.7
March	272	5.65	204	4.28	25.0	24.2
April	294	4.87	200	2.99	32.0	38.6
May	342	5.79	188	4.08	45.0	29.5
June	330	5.72	190	4.62	42.4	19.2
July	398	8.34	244	5.12	38.7	39.8
August	438	8.77	284	5.50	35.2	37.3
September	414	9.03	242	5.50	41.5	39.1
October	414	8.83	248	5.21	40.0	41.0
November	478	10.29	280	5.75	41.4	44.1
December	390	9.60	294	7.25	24.6	24.5

were used, each 166.7 ft. long, 40 ft. wide and 7 ft. deep. The average rate of flow was 16,800,000 gal. daily. If it is assumed that the sludge was 2 ft. deep, the depth of the flowing sewage would be 5 ft., and on this basis the period of detention was 0.35 hour and the average horizontal velocity was 475 ft. per hour, or nearly 7.8 ft. per minute. If the total depth of the basin is taken in estimating the rate of horizontal flow, the latter is 333 ft. per hour, or 5.6 ft. per minute. The results for a full year are given in Table 76.

At Columbus, Ohio, experiments were made with 2 tanks 8 ft. deep and 40 ft. long, with an effective capacity of about 17,000 gal., the sewage passing continuously through them. The time of detention was 8 hours in Tank A and 6 hours in Tank B, giving velocities of 4.9 ft. and 6.7 ft. per hour. The efficiencies of the tanks, supplied with sewage previously passed through grit chambers, are stated in Table 77.

TABLE 77.—RESULTS OF SEDIMENTATION TANK EXPERIMENTS, COLUMBUS, 1904–1905

(Report on Sewage Purification, page 90, Geo A. Johnson)

	Tank A, operated at 4.9 ft. per hour from Aug., 1904 to June, 1905			Tank B, operated at 6.7 ft. per hour from Nov., 1904 to April, 1905		
	Influent	Effluent	Per cent. removed	Influent	Effluent	Per cent. removed
Oxygen consumed	46.0	37.0	20	47.0	39.0	17
Organic nitrogen	8.0	6.4	10	7.6	6.3	17
Nitrogen as free ammonia	11.7	11.7	0	11.3	11.0	6
Suspended matter:						
Total	147.0	78.0	47	134.0	73.0	46
Volatile	64.0	34.0	47	64.0	38.0	41
Fixed	83.0	44.0	53	70.0	35.0	50

Further experiments with a tank 200 ft. long, 8 ft. wide and 7 ft. deep, operated with a velocity of 48 ft. per hour indicated no great difference in the amount of suspended matter at different depths, in the upper part of the tank, although there seemed to be an appreciable diminution in the amount as the bottom layers were approached. This indicated to Johnson a probable aggregation of particles as they traversed the depth of the liquid, whereby their respective hydraulic subsiding values were increased.

At Gloversville, N. Y., experiments with a settling tank 8 ft. wide, 8 ft. deep and 32 ft. long, gave the results recorded in Table 78. With regard to the high removal of suspended matter, it should be noticed that this sewage contains large quantities of tannery wastes and chemicals,

which aid in the precipitation of the suspended matter. A large proportion of the tanneries were equipped with individual settling tanks in which much of the solid matter carried by the wastes was deposited before they reached the sewers.

TABLE 78.—EXPERIMENTS WITH CONTINUOUS HORIZONTAL-FLOW SETTLING TANKS AT GLOVERSVILLE, N. Y., 1908–1909
(Parts per 1,000,000)

Month	Detention period, hours	Velocity, ft. per hour	Suspended matter in sewage		Suspended matter in effluent		Percentage removed	
			Total	Volatile	Total	Volatile	Total susp. matter	Volatile
August...	8	4	447	280	74	54	84	81
September	8	4	352	216	79	62	78	71
October...	8	4	384	229	71	52	82	77
November	8	4	392	225	75	57	81	75
December.	8	4	304	201	87	66	71	67
January...	6	5.3	294	189	79	63	73	67
February..	6	5.3	516	281	80	66	84	77
March....	6	5.3	257	151	70	64	65	58
April......	6	5.3	359	190	80	56	78	71
May......	6	5.3	554	245	104	70	81	71
June......	6	5.3	556	274	108	82	81	70
Average..	.....	......	388	230	81	61	79	74

At the Philadelphia sewage experiment station tests were made of several settling tanks using different rates of flow, allowing from 6 to 3½ hours detention period. One of these tanks was so proportioned that the length was four times the depth. As first operated it was unbaffled and had a detention period of 10 hours. Under these conditions it yielded an effluent having 65 parts per 1,000,000 of suspended matter and a removal of 50.4 per cent. In the next run the detention period was reduced to 6 hours. The effluent then contained 69 parts per 1,000,000 of suspended matter, but as the inflowing sewage was much stronger the percentage removal rose to 67.5 per cent. The detention period was next reduced to 4 hours, when the effluent showed only 44 parts per 1,000,000 or 81.2 per cent. removal. This abnormal improvement is unexplained. Baffle walls and scum boards were then introduced and a rate equivalent to 4 hours' detention maintained. Under these conditions the effluent had 54 to 67 parts per 1,000,000 suspended matter and

showed 75.8 to 71 per cent. removal. The general conclusion was reached that long detention periods are unnecessary and that great improvement in the uniformity of the tank effluent is obtained by efficient baffling, and this has been found elsewhere to be particularly true where temperature conditions are variable.

Dr. Imhoff informed the authors that he rated the detention period as of the greatest importance, and velocity through the tank as of no importance if less than 350 to 600 ft. per hour. He designated the relative importance of various factors roughly as follows:

Factor	Importance
1. Detention period	100
2. Change in temperature	30
3. Depth	30
4. Construction of inlets, outlets, etc.	30
5. Character of sewage, fresh or septic, domestic, manufacturing, etc.	30
6. Velocity through tank, if less than 350 to 600 ft. per hour	0

Relation of Strength of Sewage to Efficiency of Sedimentation.—Fig. 80 has been plotted from data relating to tanks receiving many kinds of sewage. It indicates the relation which the quantity of suspended matter in the sewage bears to the rate of subsidence and percentage of removal during different periods of time. In general it appears that the greater the quantity of suspended matter in a given volume of sewage the greater will be the percentage of reduction.

VERTICAL SEDIMENTATION TANKS

Vertical sedimentation tanks for sewage treatment were first used in Germany at Halle and Dortmund. In these tanks the sewage flows slowly upward while the sediment is deposited in the bottom. The sludge is removed by opening a valve, when the pressure of the supernatant liquid forces the sludge out through a pipe.

In such tanks the influent pipe extends to a considerable depth below the surface and admits the sewage at a relatively low velocity, distributing it as well as may be throughout the horizontal cross-section of the tank. This pipe must terminate at an elevation several feet above that to which it is intended to allow the accumulated sludge to rise, in order to avoid stirring up solid matter already deposited, unless it is desired to produce contact of the influent with the sludge for the purpose of aiding precipitation by attraction and coagulation, which Prof. Robert Spurr Weston has found advantageous with industrial wastes.

After leaving the inlet orifice, the sewage spreads out as it rises in the tank and its velocity is gradually reduced to a rate at which the particles

of suspended matter are just held in equilibrium, neither rising nor falling. As more sewage passes this zone the suspended matter is mechanically

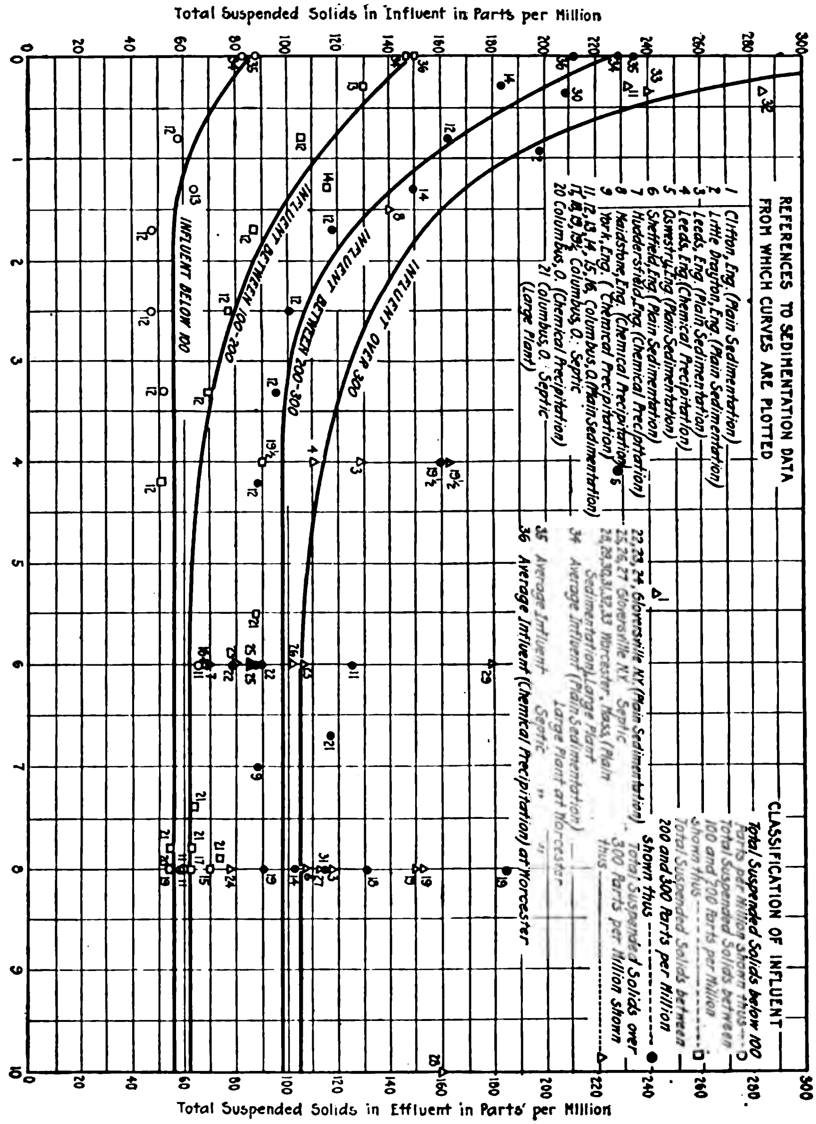


FIG. 80.—Removal of suspended solids in sewage by sedimentation.

filtered out by the floating mass, which increases in density, and coagulation occurs by the aggregation of particles. When this mass becomes

sufficiently dense, portions drop out of the zone and fall by virtue of their weight, finding their place in the sludge accumulation in the bottom of the tank. Statistics relating to some of these tanks are given in Table 79.

TABLE 79.—DATA ON VERTICAL SEDIMENTATION TANKS

Locality or authority	Kind of tank	Dimensions	Period of flow, hours	Velocity: Ft. per hour	Velocity: Mm. per second
Birmingham, Eng.	Watson	44 ft. dia. × 36 ft. 6 in. deep	4.0	4.4	0.37
Dortmund, Germany	Dortmund	21 ft. 4 in. dia. × 39 ft. deep	....	Av. 15	1.28
Dr. Dunbar, Germany	Dortmund	45 ft. deep	1¾	6.6	0.56
Essen, Germany	Dortmund	....	....	Av. 18 Max. 24	15.3 20.4
Gloversville, N. Y.	Secondary	35 ft. 10 in. dia. × 22 ft. 6 in.	1.16	8.65	0.73
	Primary	36ft. 4 in. dia. × 32 ft. 6 in. deep	3.47	8.65	0.73
Minworth Greaves, England.	Watson	....	....	7.0	0.60
Neustadt, Germany	Mairich	22 ft. deep	....	....	......
Nuneaton, England	Watson	24 ft. dia. × 34 ft. deep	7.2	4.7	0.4
Stargard, Germany	....	8½ ft. deep	2.0	6.0	0.51
World's Fair, Chicago	Dortmund	[1]32 ft. dia., × 54 ft. deep	7.32[1]	4.37	0.37

[1] Depth to bottom of cone. Period figured above cone.

Note.—An Imhoff tank in the Emscher District, 25 ft. in diameter and 30 ft. deep, has shown a removal of 75 per cent. of the suspended solids.

A typical Watson tank is illustrated in Fig. 87, page 383. The Dortmund type is shown in Fig. 85, page 381, and the Mairich type is similar to it.

In 1909 experiments were made at Gloversville, N. Y., on a vertical tank made of 28-in. sewer pipe. The outlet overflow was through four notches in the bell of the top pipe, 16 ft. above the inlet pipe. The inlet was through a 2-in. wrought-iron pipe turned up vertically about 1 ft. above the bottom of the tank. Wooden baffles were placed over this inlet to break the velocity. The tank was cleaned morning and afternoon after each test. Samples were drawn off at various depths through faucets. At each depth there appeared to be particles of a certain size in suspension just balanced by the velocity of flow and the force of gravity. The results of these tests are shown in Table 80. This sewage is radically affected by tannery wastes, as previously stated. The chemicals in them often cause a fairly good chemical precipitation and thus aid materially in the efficiency of the sedimentation process. It will be noticed that the sewage usually contained a much greater

quantity of suspended matter per unit of volume in the forenoon than in the afternoon, and that the efficiency of the sedimentation was also greater in the forenoon.

TABLE 80.—VERTICAL TANK EXPERIMENTS AT GLOVERSVILLE, N, Y., 1906

Upward veloc., ft. per hr.	Detention period, hr., min.	Total suspended matter, parts per 1,000,000						Percentage reduct. of total susp. matter	Time
		Crude sewage	At depth of				Effluent, 16 ft. depth		
			4 ft.	7 ft.	10 ft.	13 ft.			
6	2:40	676	364	352	336	324	264	61	9–10:40 a.m.
		792	392	336	244	224	196	75	9–10:40 a.m.
		312	156	112	108	92	80	74	2– 4:40 p.m.
		392	256	220	200	192	172	56	2– 4:40 p.m.
8	2:00	952	192	152	124	124	124	87	9–11:00 a.m.
		764	444	352	284	264	236	69	9–11:00 a.m.
		280	192	184	148	140	116	59	2–4:00 p.m.
		292	208	184	168	148	116	60	2–4:00 p.m.
10	1:36	852	280	216	196	188	128	85	9–10:36 a.m.
		1708	500	356	344	324	308	82	9–10:36 a.m.
		544	224	212	160	152	96	82	2– 3:36 p.m.
		244	212	184	180	156	108	56	2– 3:36 p.m.
12	1:20	944	232	212	176	168	156	83	9–10:20 a.m.
		848	460	340	284	264	256	70	9–10:20 a.m.
		372	240	184	172	164	132	65	2– 3:20 p.m.
		324	260	204	160	156	120	63	2– 3:20 p.m.
14	1:09	940	412	224	156	132	104	89	9–10:09 a.m.
		876	464	276	260	228	180	79	9–10:09 a.m.
		232	220	208	204	152	112	52	2– 3:09 p.m.
		580	240	232	220	204	148	74	2– 3:09 p.m.
16	1:00	728	.....	312	276	252	204	72	9–10:10 a.m.
		888	560	432	328	316	312	65	9–10:00 a.m.
		432	.....	.....	288	180	152	65	2– 3:00 p.m.
		348	.....	244	180	164	140	60	2– 3:00 p.m.
18	0:53	828	516	300	252	244	216	70	9– 9:53 a.m.
		588	360	256	208	196	176	58	9– 9:53 a.m.
		316	300	296	244	200	156	74	2– 2:53 p.m.
		324	.....	.....	256.	192	136	51	2– 2:53 p.m.
20	0:48	512	.....	316	284	188	152	70	9– 9:48 a.m.
		856	.....	628	532	392	268	68	9– 9:48 a.m.
		284	268	256	236	176	128	55	2– 2:48 p.m.
22	0:44	956	.....	.....	428	404	280	71	9– 9:44 a.m.
		952	.....	572	320	288	212	78	9– 9:44 a.m.
		400	.....	332	256	204	144	63	2– 2:44 p.m.
		388	.....	.....	320	208	132	66	2– 2:44 p.m.
24	0:40	592	.....	336	300	272	224	62	9– 9:40 a.m.

From Table 80, it appears that with a given velocity the efficiency increases with the height of flow, or, in other words, the deeper the tank the more efficient will be the sedimentation at a given rate of flow. A study of Table 81 indicates that greater efficiency can be obtained by increasing the vertical height through which the sewage must flow than by increasing the area of the tank and decreasing the velocity and the height through which the sewage must rise. From Table 80, it appears that with a velocity of 12 ft. per hour and a rise of 16 ft., 63 to 83 per cent. of the solids were removed. If the tank had been increased 100 per cent. in cross-sectional area, the velocity would have been reduced to 6 ft. and if at the same time the rise had been reduced to 8 ft. the proportion of suspended matter removed would have been only about 56 to 75 per cent.

Table 81.—Percentage of Suspended Matter Removed at Different Depths in Vertical Tank at Gloversville, 1909

(Averages of individual experiments)

Upward velocity, ft. per hour	Depths of sedimentation				
	4 ft.	7 ft.	10 ft.	13 ft.	16 ft.
6	46	53	59	62	67
8	37	46	55	59	65
10	56	63	67	70	80
12	52	62	68	70	73
14	49	64	68	73	79
16	37[1]	51	63	64	65
16	....	....	55[2]	62[2]	66[2]
18	32	51	59	63	68
20	....	27	36	54	67
22	....	33	57	64	74
24	....	43	49	54	62

[1] One experiment only.
[2] Average of 4 experiments.

Many experiments have been made on radial flow Imhoff tanks, in which the sewage enters in the middle at the surface, flows downward under a baffle and then upward to an outlet weir placed around the edge. Plants of this character in the Emscher District in Germany remove from 40 to 80 per cent. of the suspended solids, with from ¾ to 1-hour detention period. On the other hand an experimental tank at the Philadelphia sewage experiment station (1910) removed only 53 per cent. of the suspended matter, but in this tank the vertical sedimentation distance was only about 4½ ft.

Twenty-two vertical tanks are in use at the disposal works in Bir-

mingham, England. Sixteen of these are each 25 ft. square and 20 ft. deep, built like inverted pyramids. Sewage enters with a velocity of 1 to 2 ft. per second through an inflow pipe dipping down into the middle of the tank. As it emerges from the pipe, it spreads out laterally and rises to the surface, the normal velocity being about 7 ft. per hour. Watson has stated that the efficiency of the tanks is not materially impaired until the velocity exceeds 15 ft. per hour. These tanks remove from 60 to 80 per cent. of the suspended matter in sewage already passed through grit chambers and septic tanks ("Works of the Birmingham, Tame and Rea District Drainage Board," Watson, page 38). The 6 large tanks have the dimensions given in Table 79.

This type of tank has been considerably used in the treatment of trade wastes, particularly that from paper mills. Where used for the recovery of pulp from the comparatively clean wash water from paper machines, they are known as "save-alls." Watson found this type particularly efficient in removing finely divided suspended matter.

DESIGN OF HORIZONTAL-FLOW TANKS

The length of horizontal-flow tanks is governed by the desired velocity and detention period, the quantity of sewage to be handled, and the area available. The width should be small in comparison with the length, to promote uniformity of distribution and flow. The ratios of a number of such tanks are given in Table 82.

TABLE 82.—RATIOS OF WIDTH TO LENGTH OF SEDIMENTATION BASINS

City	Width	Length	Ratio	City	Width	Length	Ratio
Bremen, Germany....	66 ft.	525 ft.	1:8	Accrington, England..	59 ft.	102 ft.	1:1.7
Cassel, Germany.....	14 ft.	131 ft.	1:9.4	Clifton, England.....	5 ft.	39 ft.	1:7.8
Frankfort, Germany..	26 ft.	136 ft.	1:5.1	Halton, England.....	12 ft.	25 ft.	1:2.1
Giessen, Germany....	8 ft.	131 ft.	1:16	Oswestry, England...	15 ft.	70 ft.	1:4.7
Hannover, Germany..	21 ft.	131 ft.	1:7.7	Baltimore, Md.......	103 ft.	420 ft.	1:4
Mannheim, Germany.	16 ft.	157 ft.	1:9.6	Mt. Vernon, N. Y....	50 ft.	100 ft.	1:2
Salingen, Germany...	23 ft.	148 ft.	1:6.4	Reading, Pa.........	52 ft.	253 ft.	1:4.9
Triel, Germany......	20 ft.	328 ft.	1:16	N. Attleboro, Mass...	22 ft.	52 ft.	1:2.4

Theoretically, as Hazen has pointed out, the degree of sedimentation is independent of the depth. Practically, however, the basins must be of sufficient depth to prevent the settled particles being lifted and carried out with the effluent and to provide sufficient storage capacity. It may range from 6 to 16 ft., depending largely on topographical conditions.

The number of basins to be provided depends upon factors explained on page 213. It should be sufficient to afford flexibility of operation by cutting in or out one or more basins as the flow of sewage varies, so as

to maintain the flow through each basin at a rate as uniform as possible. Even small works should be provided with at least two units, to allow cleaning and repairs without interrupting service.

Basins may be operated either in series or parallel. In England some of the earlier tanks were built along three sides of a rectangle, like an angular horse-shoe. Another favorite form, and one adopted to a considerable extent in America, is a rectangular tank with one or more baffle walls extending nearly to the surface of the sewage, making virtually serial tanks. Fuller says of it in his treatise on "Sewage Disposal:"

"Within reasonable limits say for a length of flow of from fifteen to thirty times the depth, tanks arranged in series have advantages that are well defined as distinguished from tanks arranged in parallel. It appears to the author that the benefit of a serial arrangement is related substantially to means for preventing the appearance in the effluent of results of disturbance of sediment upon the bottom, due to the effect of occasional increasing velocities."

The design of the inlet and outlet connections is of great importance as any disturbance or irregularity affects the uniformity of flow through the tank, causing dead spaces where no useful work is performed. The simplest and most obvious way of effecting uniform distribution is by means of inlet and outlet weirs extending entirely across the ends of the tank. While such weirs are satisfactory for discharging the effluent, they are not very practical for introducing the sewage into the tank. This is chiefly because of the tendency of the suspended matter to settle in the influent channel and to collect on the edge of the weir. Attempts have been made to avoid this difficulty by cutting orifices in the side of the influent channel, but with generally unsatisfactory results. The best method appears to be to provide a few relatively large openings through which the sewage flows at moderately high velocity, which is checked almost immediately by small baffles placed in front of and close to the influent openings. Fig. 81 illustrates such a baffle used by the authors in a covered sedimentation tank.

Baffles and scum boards are in very general use for the purpose of maintaining uniform conditions of flow. They must be carefully arranged, for increasing the rate of flow by too much baffling will result in a mixing and general stirring up of the sediment. Scum boards extending a few inches below the surface are usually placed near the outlets, and sometimes near the inlets also, to prevent movement of the scum. Sometimes transverse baffles extending to considerable distances below the surface are placed at equidistant points throughout the length of the tank, these in turn alternating with sludge dams, 1 ft. or more high, built across the bottom of the tank. At the Philadelphia experiment station tests made on baffled and unbaffled tanks showed a marked

superiority in favor of baffled tanks. The report of the Bureau of Surveys, George S. Webster, Chief Engineer, says on this subject:

"Another matter not shown in the analyses, but noted during the operation of all of the tanks, was that in unbaffled tanks the presence of visible trade wastes at the influent end was soon followed by the same waste at the outlet, showing a current of high velocity through quiet portions of the tank, whereas after proper baffling of the tanks the entire cross-section of the tank was placed in service."

Vertical baffles have been used in a few instances to still water for measuring purposes. The nearest approach to such baffling in sewage sedimentation tanks is in the Travis tanks provided with "colloiders,"

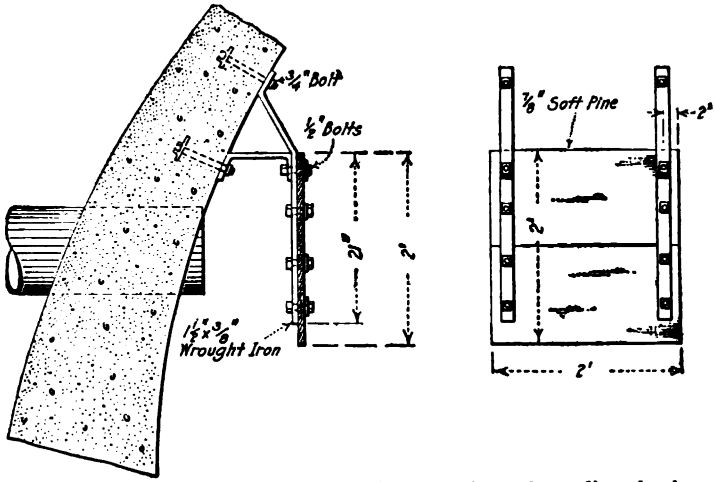


FIG. 81.—Baffle plate in front of inlet pipe of settling basin.

as at Norwich, England. As the object of baffles is to secure a uniform distribution of the stream of sewage throughout the upper portion of the tank, it may well be that vertical baffles would be effective if properly designed.

According to Leslie C. Frank and Franz Fries (*Engineering Record*, November 1, 1913), experiments in the Emscher District of Germany have resulted in the adoption of only two baffles for each Imhoff tank, one near the inlet and one near the outlet of the tank, except in the case of tanks over 100 ft. long, where an intermediate baffle would be used. These penetrate only from 12 to 16 in. below the surface of the liquid and are scum boards rather than baffles. It has been concluded that no attempt should be made to promote a uniform velocity throughout the cross-section of the tank. The engineers of the District see no

reason why the velocities in the upper part of the tank should not be higher, within limits, than those in the lower part of the tank. It is simply necessary, they assert, for the upper currents to have sufficient time to drop their load of suspended matter beyond the pull of the outlet device. Baffles have been tried, both in the upper and lower parts of tanks, but it has been found that they not only reduce the cross-section of the tank but that they produce subcurrents, thereby hindering rather than promoting sedimentation. It has been found by coloring the inflowing sewage with dye that some of the liquid passes through the tank in much less than the estimated time of detention. The Emscher district engineers do not consider this important, however, because the parts of the liquid with the highest rate of flow are near the surface and before they reach the outlet they deliver their load of settling solids to the more slowly moving sewage lower in the tank.

In a good many small and medium-size plants the sedimentation basins have been roofed over, in some cases on account of severe winter temperatures and in others to conceal the tank and sewage from view. There is some increase in efficiency of covered tanks, due to protection from wind and from temperature changes, but the effects of these alone are not generally of such importance as to warrant the construction of roofs. Where tanks are covered in the ordinary manner it is not as easy to inspect them and note their condition, which is a disadvantage. Many of the larger works, such as those at Worcester, Columbus, and Baltimore, have open tanks. Fig. 82 is a view of the Worcester installation. Open tanks should have walls extending at least 6 in. above the surface of the liquid in order to prevent the latter from being blown over their coping during windy weather. They should also be surrounded by fences if children can have access to them.

A covered sedimentation basin built from the plans of the authors is illustrated in Fig. 83. This has a capacity of 322,000 gal. and cost complete $13,538. It consists of 4 compartments, *A* and *B*, and *C* and *D*, each pair forming practically serial tanks. The flow of sewage to this plant fluctuates widely on account of the infiltration of large amounts of ground water. The plant was designed with this circumstance in mind, so as to allow for considerable flexibility in the method of operation. For example, if compartments *C* and *D* are being used for sedimentation it is possible to utilize as a dosing tank compartments *E*, or *B* and *E*, or *A*, *B* and *E*.

When a tank is to be cleaned the supernatant sewage is drawn off by means of a pipe extending from the bottom to the top of the basin and provided at the bottom with a swivel joint (Fig. 84), so that it can be revolved through an angle of 90 deg. To the top of this pipe is attached a float of sufficient size to prevent the pipe from falling to the bottom, in which case it would draw sludge. The float is so adjusted as to hold

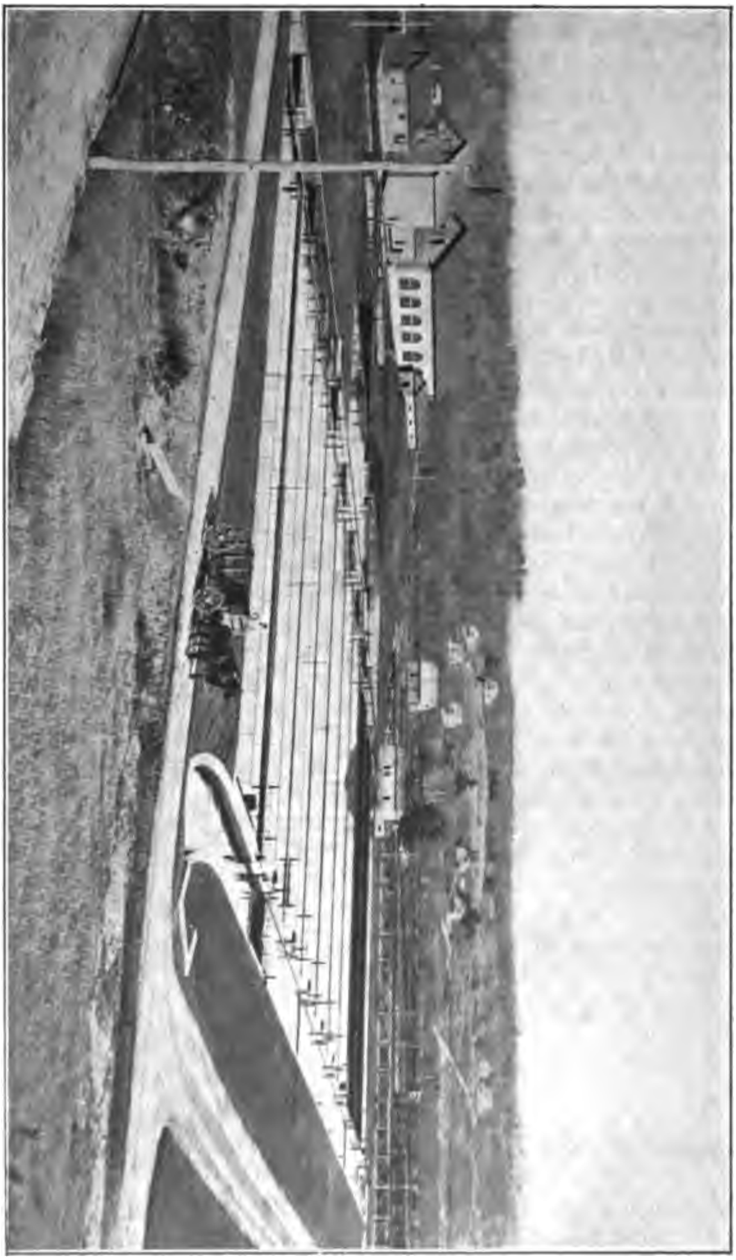


Fig. 82.—Sedimentation tanks at Worcester, Mass., sewage treatment works.

the pipe a few inches below the surface of the sewage as it is drawn down, and a stop is provided so that when the pipe has fallen to a predetermined point it can go no farther, thus preventing its falling down into

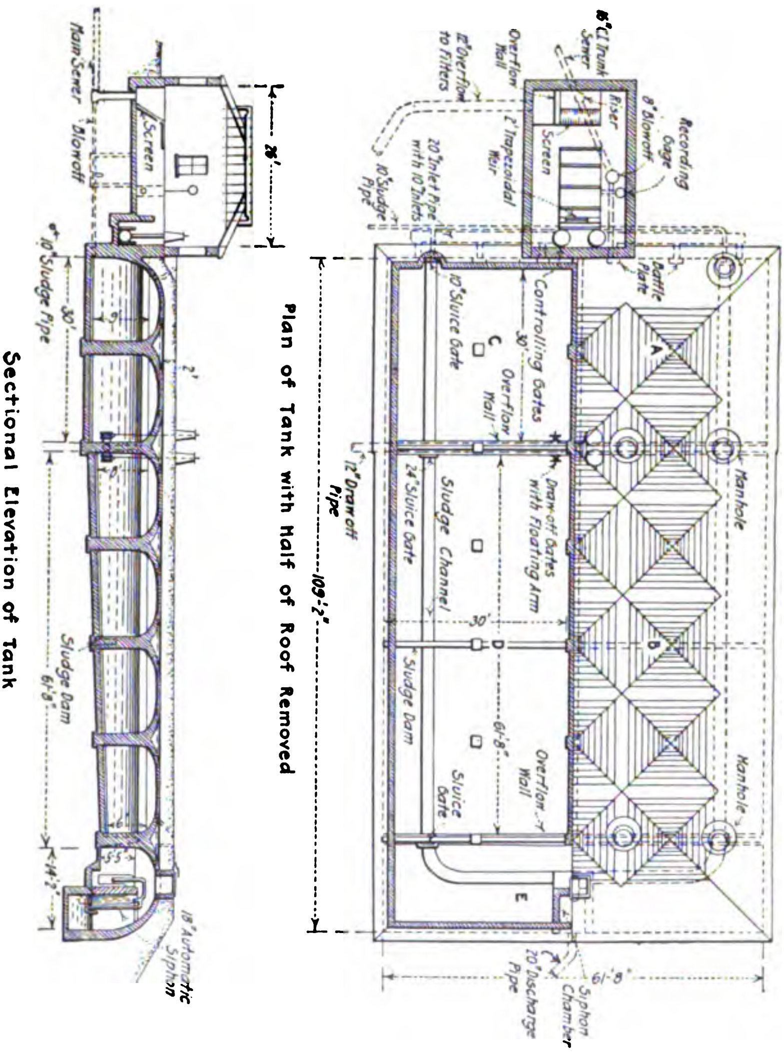


FIG. 83.—Sedimentation basin at Marlboro, Mass.

and drawing the sludge. A gate valve should be provided on the pipe line with which the swivel pipe is connected so that the upright pipe can be kept partly filled, because its buoyancy will throw it up out of the

water if empty. The upper end of the pipe should be enlarged and cut back at an oblique angle so that as it falls the opening will gradually approach a horizontal position and the sewage will be drawn from the surface. Four of these draw-off pipes cost $765, complete, in place in the basin illustrated in Fig. 83. The conditions governing the slopes of floors are explained in Chapter XII, on Sludge.

DESIGN OF VERTICAL-FLOW TANKS

The form of construction used for vertical-flow tanks lends itself particularly to the removal of sludge without drawing off the supernatant liquid, and they are invariably built with this end in view.

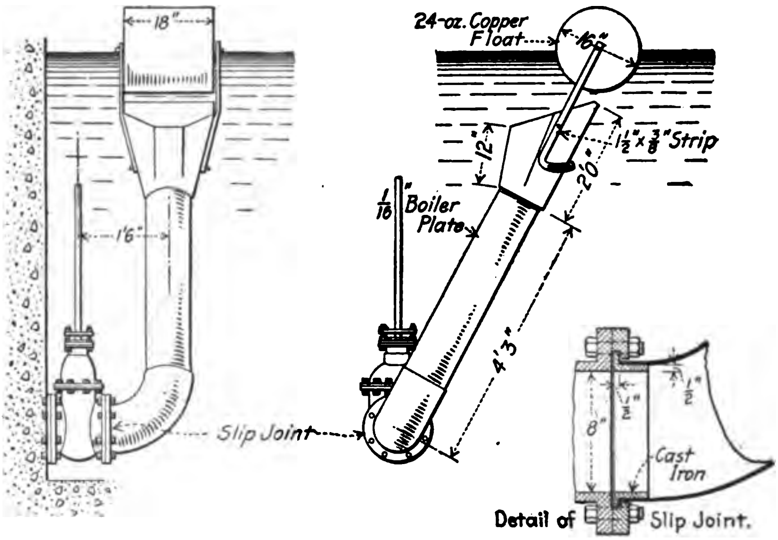


FIG. 84.—Swiveling outlet pipe for settling basins.

The first tanks of this type built in America were those at the Columbian Exposition at Chicago in 1893. These were modeled after those in use at Dortmund, Germany, and are shown in Fig. 85, from *Engineering News*, August 3, 1893.

The sewage passed down through central cylinders 6 ft. in diameter to points near the bottoms of the tanks, where it was distributed by sheet-iron arms and the precipitate settled to the bottom. The clarified liquid rose slowly to the top, where it was removed by a set of troughs drawing as nearly as possible equal quantities of effluent from equal areas of surface. The sludge was drawn off from time to time without interruption to the continuous action of the tanks.

Vertical-flow tanks are particularly adapted to the clarification of the effluent from trickling filters, because the suspended matter in such effluents, if allowed to accumulate on the sides and bottom of very shallow tanks, will reduce the quantity of dissolved oxygen in the liquid passing through them. This makes it desirable to reduce as much as possible the time of contact of the liquid and sludge, which is best accomplished in vertical-flow tanks.

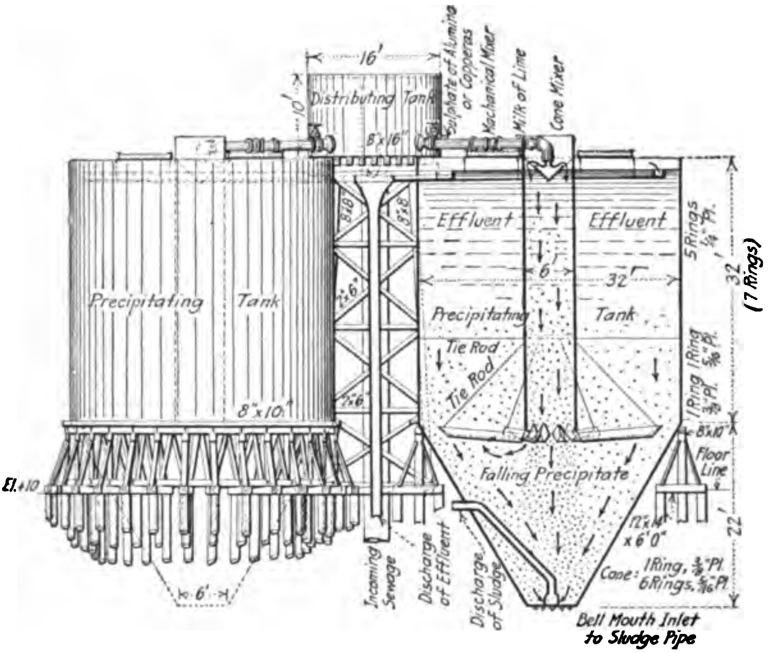


FIG. 85.—Vertical-flow tanks at Columbian Exposition.

The Birmingham tanks differ principally from the original Dortmund type in that the vertical portion is relatively very low, only about half of the height of the conical part. John D. Watson, their designer, thinks that inasmuch as the velocity in the vertical part is uniform there is no merit in making it of any considerable height, which is contrary to the deductions from the Gloversville investigations, page 373. Sewage enters a Birmingham tank through a pipe carried to the center of each tank and pointed downward, and spreads out laterally, rising with a normal velocity of about 5.5 to 9.8 ft. per hour, being somewhat higher in the silt tanks than in the separating tanks receiving filter effluent. The effluent is discharged over weirs extending for the full length of one side of each of the tanks.

As a result of the experiments outlined earlier in this chapter, vertical-flow tanks have been built at Gloversville, N. Y., for both preliminary sedimentation and the clarification of trickling filter effluents. The two primary tanks are shown in Fig. 86 and in them the normal vertical velocity is about 8 ft. 6 in. per hour. The secondary tanks are similar, except that the vertical portion is only 11 ft. deep.

The sedimentation tanks at Toronto, Ont., were built from plans approved by Watson, and the plant is designed to handle 240 U. S. gal.

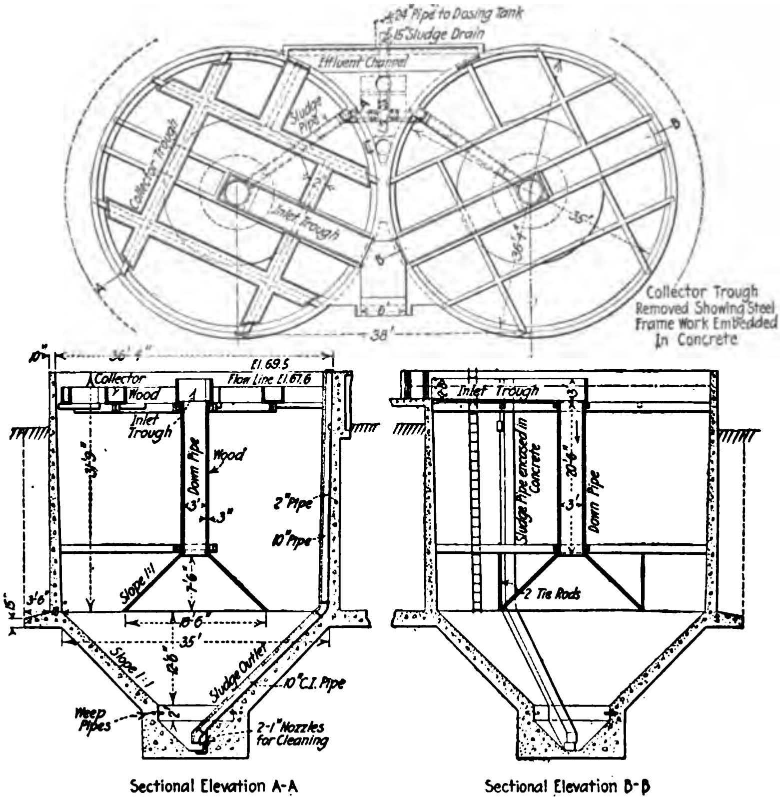
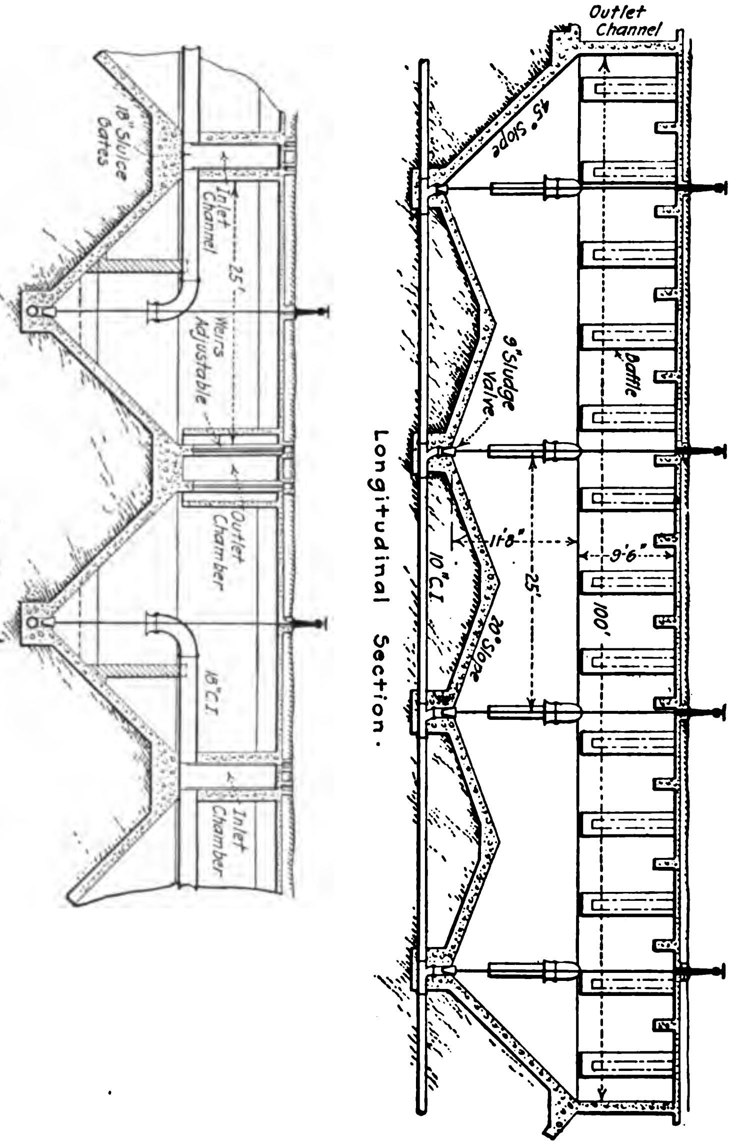


FIG. 86.—Vertical-flow settling tanks, Gloversville, N. Y.

per capita daily from a population of 550,000. The sewage first passes through the screening plant described on page 318, and then enters the covered tanks, Fig. 87. There are 24 of these tanks, each 25 ft. wide and 100 ft. long in the clear, with 4 hoppers in the bottom. These tanks are arranged in a block about 207 ft. wide and 366 ft. long. The flow line is about 16 ft. above the lowest point of the hopper, and the capacity of a



Transverse Section

Fig. 87.—Vertical-flow settling basins, Watson type, Toronto, Ont.

tank filled to this elevation is about 180,000 gal. Commissioner Harris of the Department of Works informed the authors early in 1915 that two typical analyses gave the following results:

Test No.	Solids in raw sewage			Solids in effluent			Removal of suspended solids, per cent.
	Total	Suspended	Dissolved	Total	Suspended	Dissolved	
1	1120	400	720	810	190	620	52.5
2	750	170	580	670	90	580	47.0

The cost of vertical-flow tanks is in general considerably greater than for the horizontal-flow type, largely on account of the depth of excavation necessary for their installation. In a report to the trustees of the Sanitary District of Chicago, made in 1911 by G. M. Wisner, the following comparative figures as to the costs of different kinds of sedimentation basins, are given:

"Comparative figures of an approximate nature are submitted here, Table 83, on the different types of settling tanks. The quantities are based on the design used for Emscher (Imhoff) tanks at Atlanta, and the vertical settling tanks of the Dortmund type used at Gloversville, and the rectangular straight flow settling basins, comparatively shallow, as used at Columbus, Ohio, in order to see how much greater is the cost of the Emscher type of tank. Excavation is included from the top of the tank down."

TABLE 83.—COMPARATIVE COSTS OF TANKS FOR SAME SERVICE
(Report on Sewage Disposal, Sanitary District of Chicago, G. M. Wisner, page 42)

Type of tank	Nominal period of settling	Gallons per capita daily	Cost per capita
Emscher (Imhoff)	3 hours[1]	200	$1.44
Dortmund	4 hours[1]	200	0.84
Straight flow	8 hours	200	0.77
Straight flow	6 hours	200	0.58

[1] In both these periods the sludge storage is not calculated in determining the nominal period of settling.

STRAINERS OR ROUGHING FILTERS

Strainers or roughing filters made of coke were installed in England at various places in the early eighties. These devices, as their name implies, were simply intended to strain out the suspended matter in the sewage. In 1886, at small sewage works in Medfield, Mass., sewage was passed upward through a layer of excelsior held in place by wooden slats.

The Massachusetts State Board of Health has experimented since 1894 with strainers made of various materials. During 1894 a strainer of coke breeze 6 in. in depth, received sewage at the rate of 1,000,000 gal. per acre per day. This removed 48 per cent. of the organic matters, with an expenditure of 10 cu. yd. of coke breeze per 1,000,000 gal. filtered. Later, screened coke was used instead of coke breeze. This strainer was operated during 1900 at a rate of 1,000,000 gal. per acre daily, with a removal of clogged coke amounting to only 0.4 cu. yd. per 1,000,000 gal. strained. The amount of organic matter retained by the strainer was 44 per cent., figured on the albuminoid ammonia determinations. The removal of suspended matter, however, was much better in the

FIG. 88.—Bottom of roughing filter, Salford, England.

strainer of coke breeze, amounting to 74 per cent. as against 59 per cent. in the strainer composed of screened coke.

Experiments were also made with fine bituminous and anthracite coal, which removed from 56 to 60 per cent. of the organic matter in suspension, at an expense of approximately 1 cu. yd. of coal per 1,000,000 gal. filtered. Also, owing to the greater specific gravity of coal, it did not have the same tendency to float as did coke, and for this reason the clogging material remained nearer the surface, requiring in general the removal of a smaller quantity than was the case with coke.

In 1901 strainers having an area of half an acre, composed of coke

breeze 8 in. in thickness, supported by layers of broken stone, were built at Gardner, Mass. These strainers were constructed in 4 units and included automatic devices for alternately filling and emptying each unit somewhat as contact beds are operated. This automatic apparatus, however, did not operate satisfactorily. The strainers have always given much trouble, as they clogged rapidly and could not be cleaned during the winter. When in operation they remove from 50 to 66 per cent. of the organic matter, as indicated by the albuminoid ammonia.

Strainers were tested at the Columbus Sewage Experiment Station in 1905, with results similar to those obtained elsewhere. Odors of sulphureted hydrogen were quite pronounced in the vicinity of the strainers, and it was also found that their efficiency fluctuated considerably. This latter effect was due to the sewage breaking through the clogged surface at some points and causing a sudden rush of sewage at a high rate until stopped by the clogging of the lower strata of coke.

Fig. 88 is reproduced from a photograph of a roughing filter in use at Salford, England, where this process was studied experimentally at an early date under the direction of Joseph Corbett, City Engineer. The plant consists of 6 filters, each divided into 3 bays of 105 sq. yd. each. The filters are composed of 3 ft. of fine gravel of $\frac{3}{16}$ to $\frac{5}{16}$ in. diameter and are underdrained through perforated tile held above the bottom by legs 4 in. high. Provision is made for reversing the direction of the flow and for introducing air through grids of pipe laid near the bottom. These filters are washed and "blown" once a day or oftener when the flow of sewage is large.

AERATION AND ACTIVATED SLUDGE

Although the aeration of sewage to hasten the oxidation of organic matter was investigated by a number of engineers and chemists from the time of Dr. Angus Smith, who reported on it in 1882 to the Local Government Board, down to 1910, it was not until Col. W. M. Black and Prof. E. B. Phelps carried out a series of experiments in that year that the possibility of improving the degree of sedimentation by aerating sewage in tanks received serious attention in the United States. The work previously done in this country had been in connection with filter beds rather than tanks. The experiments of Black and Phelps indicated that a considerable reduction in putrescibility could be accomplished by forcing air into sewage in basins.

Massachusetts State Board of Health.—In 1912, an investigation of the effect of aeration was started at the Lawrence Experiment Station by H. W. Clark and Stephen DeM. Gage. According to the 1913 report of the work of the station, the early experiments were made by forcing air by a Richards pump into sewage in gallon bottles. The

plain aeration treatment was carried on at first by forcing air into the sewage for 24 hours and then allowing it to settle for 1 hour. Care was taken to clean the bottles as soon as green growths appeared on the glass. When the simultaneous action of algæ and aeration was studied, the aeration was continued for 24 hours and the sample was then allowed to stand for 24 hours to allow the algæ, mainly Scenedesmus and Protococcus, to develop. The sewage which was submitted to plain aeration was called "aerated," and that which was submitted to aeration and the action of algæ was called "inoculated." The experiments during the first part of 1912 were made with strained sewage, and showed great reductions in free ammonia, total and dissolved Kjeldahl nitrogen, and oxygen consumed, as recorded in Table 84.

TABLE 84.—EFFECT OF AERATION AND OF AERATION AND INOCULATION UPON SEWAGE IN BOTTLES

(Report Mass. State Board of Health, 1913, page 296)

Sewage	Hours of		Percentage reduction in			
	Aeration[1]	Settling	Free ammonia	Kjeldahl nitrogen, Total	Kjeldahl nitrogen, In solution	Oxygen consumed
Plain aeration:						
Strained	24	1	32	37	45	43
Settled	24	1	31	61	61	64
Settled	12	1	24	67	67	63
Settled	8	....	14	42	32	48
Settled	6	1	20	24	26	34
Aerated and inoculated:						
Strained	24	24	44	23	50	43
Settled	24	24[2]	39	66	67	64
Settled[3]	24	0	52	59	55	59

[1] Volume of air used was unlimited.
[2] Extra 24 hours allowed for growth of algæ during first 2 months of this 5-month period.
[3] Algæ growth very feeble.

During the latter part of 1912, settled sewage was used instead of strained sewage, and it was discovered unexpectedly that nitrification was taking place in the bottles, with both aerated and inoculated sewage. During the latter part of this stage of the investigation it was found to be unnecessary to allow 24 hours' rest for the growth of algæ. The inoculated sewage generally showed a greater degree of purification than the aerated sewage, as indicated in Table 85, attributed by Clark and Gage to the much greater nitrification in the former. During the greater part of 1913 the culture of algæ used in seeding the sewage became less vig-

orous and nitrification was much less active in the inoculated samples. The heavier the green growth on the sides of the aerating bottles, the clearer the aerated sewage, which compared favorably at times with the effluents from sand filters.

TABLE 85.—EFFECT OF AERATION OF RAW SEWAGE IN TANK

(Report Mass. State Board of Health, 1913, pages 296, 298)

Aeration		Percentage reduction in				Average solids in effluent, parts per 1,000,000		
Hours	Cu. ft. per mil. gal. per hour	Free ammonia	Kjeldahl nitrogen Total	Kjeldahl nitrogen In solution	Oxygen consumed	Total	Loss on ig.	Fixed
24	200,000	7	41	30	59	......	......	......
10	200,000	20	56	27	61	39.5	11.8	27.7
7½[1]	200,000	3	51	7	41	40.9	11.8	29.1
6	200,000	8	42	25	51	45.7	12.2	33.5
5	100,000	15	37	29	41	41.1	9.5	31.6
5	25,000	13	56	35	44	45.7	13.3	32.4

[1] Aeration for 8 hours during 8 days and for 7 hours during 8 days.

The clarification of the sewage by aeration and the action of algæ was so noteworthy that an experiment on a larger scale with raw sewage was undertaken in 1913. This was made with a tank containing a pile of horizontal sheets of roofing slate with clear openings of 1 in. between them. A perforated pipe on the bottom of the tank was supplied with air by a blower. The aeration of the sewage was carried on continuously for 24 hours, during the first period, and the period was subsequently reduced to 10, 8, 7, 6, and 5 hours. For the earlier part of this set of tests the volume of air used was about 200,000 cu. ft. per hour per 1,000,000 gal. of sewage, and for the latter part it was about 100,000 cu. ft. When these tests were completed, the slates were washed and placed vertically 1 in. apart. In washing them about ½ in. of well-fermented, inoffensive sludge was removed. After reconstruction, the tank was aerated for 5 hours with about 150,000 cu. ft. of air, and later this amount was cut down to 25,000 cu. ft. per hour per 1,000,000 gal. With this low amount of air, the sewage entering the tank was aerated by dropping on a splash-plate distributor, so that it might have some dissolved oxygen when the air-blast was started.

The efficiency of the tank at no time equalled that of the bottles, the best work being done with 10-hour aeration with 200,000 cu. ft. of air per hour per 1,000,000 gal., as shown in Table 85. Clark and Gage attributed this superiority of 10-hour aeration over 24-hour aeration to the failure to form a gelatinous film on the slates during the 5 weeks of 24-hour aeration. This opinion was confirmed by similar results after

cleaning and placing the slates upright, the reduction in Kjeldahl nitrogen during the first part of the period of 5-hour operation being only one-fifth of the reduction during the latter part of the period, when the gelatinous film had been established again. Concerning this film and the sludge in the tank Clark and Gage make the following statement:

"Soon after the tank was put into operation the slates and sides of the tank became covered with a heavy brownish-gray growth of a gelatinous consistency, which appeared to collect mechanically the suspended matters and a large part of the colloidal matters of the sewage. After aeration was stopped, clear sewage could be drawn from the tank immediately, the suspended matters being practically held by the growth upon the slates. Without taking into consideration the value of oxidation, which is amply shown in the results of the filtration of this sewage, the process as carried out in this tank produced a much better clarification of the sewage than did any of the clarification processes operated at the station with the single exception of the one in which the sewage was precipitated with excessively large quantities of copperas and lime. The smallest average removal of total and organic matters in suspension was over 77 per cent., and during the earlier portions of the experiment when air was applied at a high rate for from 10 to 24 hours, the removal of suspended matters averaged over 90 per cent. During the last 4 months of the year, when only 25,000 cu. ft. of air per 1,000,000 gal. of sewage was applied for only 5 hours, the removal of total and organic matters in suspension averaged 82 and 80 per cent. respectively.

"With the slates in horizontal position, it was necessary to remove the accumulated matters from them by flushing at intervals, but during the latter portion of the experiment with the slates in a vertical position the jelly-like masses sloughed off from time to time in large flakes and settled almost immediately to the bottom of the tank. The sludge which is contained in the last inch or so of sewage in the bottom of the tank has been run to waste in these experiments. This sludge, owing to the fact that it is formed under strictly aerobic conditions, is inoffensive, of much lower water content than the sludge from other clarification processes, and resembles quite closely the sediment discharged from trickling filters. In practice it would be possible to dispose of this sludge with much less trouble than is the case with the sludges resulting from the usual preliminary treatments." (Rept. Mass. St. Bd. Health, 1913, page 294.)

The depth of sewage in the preliminary tank was about 15 in., but Clark and Gage consider deeper tanks more economical in most practical cases because the greater cost of forcing air through a greater depth of sewage would probably be offset by a reduction in the cost of construction and the aeration of a much larger volume of sewage by a given volume of air. Assuming a working depth of sewage of 5 ft. and electricity at 4 cts. per kilowatt-hour, it would cost about $1.85 to treat 1,000,000 gal. of sewage for 5 hours with 25,000 cu. ft. of air per hour, according to the report.

The stability of the tank effluent was never equal to that of the bottle experiments and there never was any appreciable amount of nitrification in the tank. The progressive effect of the aeration was determined by analyses of samples drawn hourly after beginning the aeration. Two sets of results of such analyses are given in Table 86. The characteristic feature of the aeration is a progressive reduction in albuminoid ammonia and oxygen consumed for about 5 hours and little or no reduction in the free ammonia until after several hours.

TABLE 86.—PROGRESSIVE CHANGES IN SEWAGE DURING AERATION

Sewage treated	Start, parts per mil.	Percentage of reduction after aeration for					
		1 hr.	2 hr.	3 hr.	4 hr.	5 hr.	6 hr.
Weak sewage							
Free ammonia.........	14.0	0	0	0	7	14	29
Albuminoid ammonia..	5.7	16	35	46	53	58	58
Oxygen consumed.....	38.0	21	38	49	55	58	59
Average sewage							
Free ammonia.........	47.0	0	0	0	0	0	4
Albuminoid ammonia..	4.6	5	13	26	41	44	44
Oxygen consumed.....	37.0	30	41	45	47	51	51

Effluents from the bottle experiments were applied to sand filters and the tank effluent to a trickling filter. In every case the rates of filtration which could be maintained steadily after the preliminary ripening of the beds were extraordinarily high, 350,000 to 450,000 gal. per acre daily with sand of 0.25 mm. effective size and 10,000,000 gal. per acre daily with stone 1 to 2 in. in size.

Manchester Experiments.—Experiments were begun in Manchester by Fowler and Mumford (*Jour. Roy. San. Inst.*, Nov., 1913) in connection with an investigation of the action of an organism named M7, which precipitated ferric hydroxide from solutions of iron salts. They found that sewage containing this organism was clarified thoroughly by aeration for 6 hours, and the effluent was non-putrefactive. The investigations at Manchester were continued by Ardern and Lockett (*Jour. Soc. Chem. Ind.*, May 30, 1914) who discovered that the sludge played an important part in the results obtained by aeration. At the outset, it was necessary to aerate the sewage samples continuously for 5 weeks before complete nitrification was attained. The clarified sewage was drawn off and a fresh sample of raw sewage was used with the old sludge below it. By repeating this process a number of times, none of the sludge being removed, the time for oxidizing fresh sewage was finally reduced

to 24 hours. The sludge accumulated in this manner was called "activated sludge." It is described by the investigators as follows:

"Activated sludge accumulated in the manner previously described is quite inoffensive, dark brown in color, and flocculent in character, and despite its low specific gravity separates from water or sewage at a rapid rate. After prolonged settlement the activated sludge, however, rarely contains less than 95 per cent. of water. A remarkable separation of the water from the sludge can be readily obtained by treatment on fine grade strainers, with the production of a sludge of the consistency of a stiff jelly. Gelatine counts have shown a bacterial content of at least 30,000,000 organisms per cubic centimeter. In addition, the sludge, by reason of its nitrifying power, must of necessity contain a large number of nitrifying organisms. It should also be noted that a fairly large number of a variety of protozoa are to be found. . . . . It does not, however, contain any algæ growths. The chemical analysis of an average sample of the activated sludge is as follows: organic matter, 64.7 per cent.; mineral matter, 35.3; total nitrogen (N), 4.6; phosphate (P_2O_5), 2.6; matter extracted by CCl_4, 5.8. Attention should be called to the abnormally high percentage of nitrogen as compared with ordinary unoxidized sewage sludge. . . . . The activity of the sludge is greatly diminished when working on the fill-and-draw method, if it is called upon to treat further samples of crude sewage prior to the complete nitrification of the previous sample dealt with. This difficulty may be overcome by simple aeration of the sludge alone until the free and saline ammonia content is removed."

In order to obtain the best results, three conditions had to be fulfilled. The first was to keep the alkalinity of the sewage rather more than equal to the nitric acid resulting from the nitrification of the ammonium salts, which occasionally made it necessary to add a small quantity of alkali before complete nitrification. The second condition was to keep the activated sludge in intimate contact with the sewage during aeration. The third condition was to adjust properly the quantity of activated sludge to the quantity of sewage, for while the reduction in the amount of oxidizable matter was not seriously affected by the varying proportions of activated sludge, the amount and rate of nitrification were influenced in a marked degree.

Experiments conducted with 1 volume of activated sludge and 4 volumes of Manchester sewage showed that, under the experimental conditions, aeration for 6 to 9 hours, with subsequent settling, was sufficient to obtain an effluent equal to that of the most efficient bacterial filters, measured by albuminoid ammonia and the 4-hour oxygen absorbed tests. During the first 3 hours, the action was mainly oxidizing and during the last 3 hours it was mainly the nitrification of the ammonium compounds. An attempt to carry out the process in two entirely separate stages was only partly successful, but the investigators were not convinced that such separation was impracticable. In experiments to ascertain the effect

of changes in temperature on the action of activated sludge, it was found that when the temperature was below 10°C. (50°F.) constantly, a very marked deterioration in the results was observed, especially with regard to the removal of colloidal matter. With the temperature between 10° and 25° (77°F.), the action was not injuriously affected, but when it rose above 30° (86°F.) there was a deterioration in the effluent.

Illinois State Water Survey.—The English work is now (1915) being supplemented by the investigations of Bartow and Mohlman in the laboratory of the Illinois State Water Survey, at Champaign, using the fresh, fairly strong domestic sewage, without trade wastes, of that college town. It has been found possible to nitrify sewage completely by blowing air

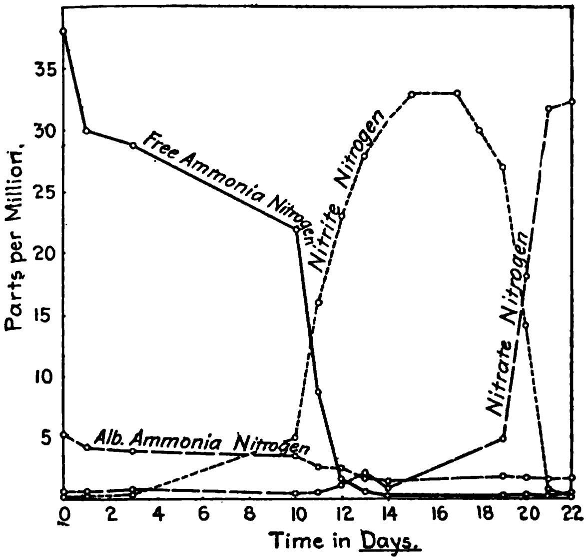


FIG. 89.—Nitrification of aerated sewage, no activated sludge present.

into it for 15 to 33 days, depending largely upon the thoroughness with which the air was distributed through the sewage. Fig. 89 shows the changes while blowing air into 3-gal. bottles, and Fig. 90 the changes when the air was blown into the space below a porous plate near the bottom of a tank 9 in. square and 5 ft. deep. In each case the free ammonia nitrogen was almost quantitively changed to nitrite nitrogen, and the nitrite nitrogen was then changed almost quantitively to nitrate nitrogen. It took 4830 cu. ft. of air for the formation of nitrate in 15 days in this way.

A great change took place when the sludge deposited from aerated sewage was left in the bottle or tank at the close of an experiment and

sewage was aerated in the presence of this activated sludge. In a paper before the American Chemical Society, April, 1915, Bartow and Mohl-

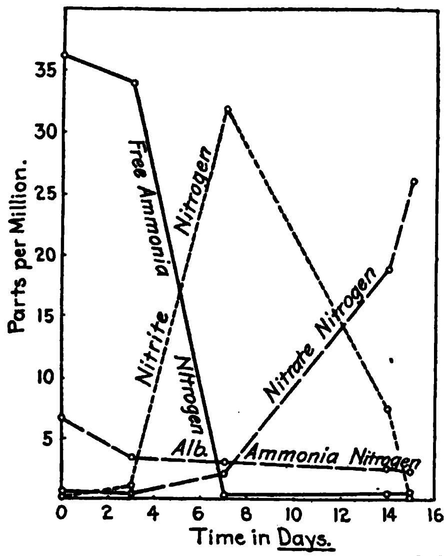


FIG. 90.—Nitrification of aerated sewage, no activated sludge present, with uniform distribution of air through porous plate.

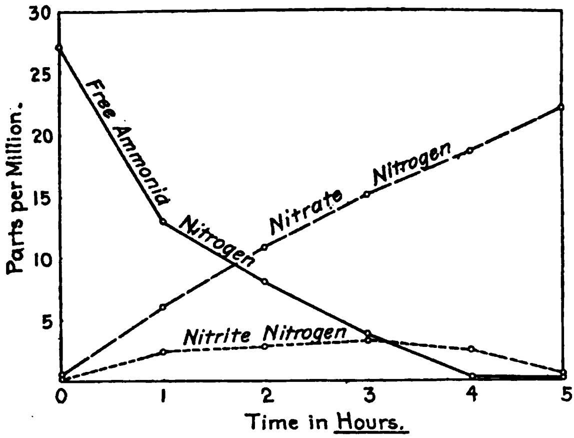


FIG. 91.—Nitrification of aerated sewage, with activated sludge present.

man stated that if such sludge from an initial aeration was left in the tank, its presence reduced the time for complete nitrification for a second treatment from 15 to 4 days and the amount of air from 4830 to 1270

cu. ft., and 34 parts per 1,000,000 of free ammonia nitrogen produced 23.8 parts per 1,000,000 of nitrate nitrogen. In the third treatment, nitrification was complete in 2 days and but 720 cu. ft. of air were used; 33 parts of free ammonia nitrogen produced 22.3 parts of nitrate nitrogen.

In the twelfth treatment the nitrification was complete in less than 8 hours with the use of less than 128 cu. ft. of air; 36 parts of free ammonia nitrogen produced 29.5 parts of nitrate nitrogen. The hourly log of the thirty-first treatment, when the tank contained five times as much sewage as activated sludge, is given in Fig. 91. For this treatment 35 cu. ft. of air were used. A sample of the sewage at the end of the first hour of aeration in the thirty-first treatment did not decolorize methylene blue at the end of 12 days, although nitrification was then incomplete. This apparently indicates that complete nitrification is unnecessary in a stable effluent. Fig. 91 shows that, in the presence of a considerable amount of activated sludge, both nitrates and nitrites are formed simultaneously by aeration.

The activated sludge lost 95.54 per cent. by drying on a water bath and then for 3 hours in an oven at 100°C. The dried material, according to Bartow and Mohlman, contained 6.3 per cent. nitrogen, 4 per cent. fat, 1.44 per cent. phosphorus, equal to 3.31 per cent. P_2O_5, and 75 per cent. volatile matter lost on ignition. It was found to be a good fertilizer.

Biological examinations showed that the sludge contained many Vorticella and Rotifera, but the predominant organism was Æolosoma hemprichi, a slender annelid worm 0.08 to 0.2 in. long. These worms feed greedily and almost continuously on any small organic particles, and presumably destroy more than their own weight of organic matter daily. Because of their method of reproduction by fission, extensive colonies are developed rapidly.

Milwaukee Experiments.—In October, 1914, experiments were undertaken by T. Chalkley Hatton, Chief Engineer of the Milwaukee Sewerage Commission, with the advice of Dr. Fowler. Both the Clark slate colloider tank and the Fowler system of aeration were under investigation at the time this chapter was sent to press, and the comments of Mr. Wm. R. Copeland, Chief Chemist, on the results obtained up to that date (April, 1915) were as follows:

"The data . . . indicate clearly that the Clark slate colloider clarifies and purifies the sewage, and that Fowler's treatment of sewage with activated sludge, as carried out in a warm laboratory, removes practically all turbidity from sewage, produces from 15 to 30 parts of nitrates, forms a granular sludge that settles readily, and reduces the number of bacteria in the sewage to the extent of 96 per cent. or more. Moreover, the sludge becomes activated more readily at a temperature of 70°F. than below 45°F."

CHAPTER XI

TANKS FOR SLUDGE DIGESTION

There are three leading types of tanks in which the solids which settle from sewage are submitted to anaerobic decomposition, and several other types have been proposed and a few of them constructed. The first of these is the septic tank, which does not differ essentially from a sedimentation basin in design, but is distinguished from it in operation by retaining the sludge until anaerobic decomposition has modified its character. This action is explained on page 213. As a result of it, the gases of decomposition lift particles of the sludge into the supernatant liquid, sometimes making it more turbid than the liquid in the same tank operated as a sedimentation basin. In some cases the quantity of sludge lifted in this way is so great that it forms a scum over part or all of the tank. Molds and fungi often develop in this mass, binding it together, and eventually even weeds may grow over its surface. At Ilford, England, the scum on a tank became 3 ft. thick, causing a material reduction in the operating capacity of the basin. Although such reduced capacity is a disadvantage, English experience favors scum, particularly in open tanks, as at least a partial preventive of odors and a cheap means of retaining the heat of the sewage in cold weather. As the desired bacterial action on the sludge is checked by low temperatures, the latter advantage of scum may be important in some localities, and has sometimes led English engineers to aid the formation of scum by floating light wood lattice frames on the surface of the liquid.

The second type of tank for sludge digestion is the Travis, shown in Fig. 19, page 216. The effluent from the tank is a mixture of relatively fresh sewage from the side chambers and of septic sewage from the middle chamber.

The third type of tank for sludge digestion is the Imhoff, Fig. 19. There is no flow of sewage in its lower chamber, which acts solely as a sludge receiver. The slots through which the sludge falls into it are so designed that the gases of decomposition cannot escape through them into the sewage flowing through the side chambers, and the processes of sedimentation and of sludge digestion are carried on as if they were conducted in wholly independent basins.

SEPTIC TANKS

In so far as a septic tank acts as a sedimentation basin, the principles of design outlined in Chapter X apply to it. They are modified, however,

by the special conditions introduced by the digestion of the sludge and the resulting characteristics of the sewage in the tank. The leading factors to be considered in design are:

1. Character of sewage reaching the treatment works.
2. Number of tanks necessary to give the proper time of detention of the sewage over the sludge.
3. Covering the tanks.
4. Inlets and outlets which will not be affected by scum.

Influence of Character of Sewage Reaching Works.—There is apparently little difference between sedimentation and septic tanks in the removal of suspended solids, in many cases. Experiments confirming this opinion were conducted by the Royal Commission on Sewage Disposal, with results given in Table 87. The experiments were conducted with a settling tank holding 6000 gal. and a septic tank holding 6600 gal., the rate of flow being 6000 gal. in 24 hours in each case. George A. Johnson has reported that parallel experiments with settling and septic tanks resulted in an average reduction in suspended matter of about 63 per cent. in sedimentation tanks with 6 hours detention and 66 per cent. with 8 hours detention, while in septic tanks with detention periods of 8, 16 and 24 hours the removal of suspended matter averaged 61, 66 and 67 per cent., respectively. ("Report on Sewage Purification," Columbus, 1905.) These experimental figures agree well with the results of the operation of the septic tanks subsequently built at Columbus. In the annual report on these operations for 1913, the removal of suspended matter during an average detention period of 4 hours is given as 60 per cent.

As in most branches of sewage treatment, this general statement is subject to exceptions, as shown by the large-scale experimental work of Eddy and Fales at Worcester, Mass., where sedimentation for nominal detention periods of 7 to 24 hours reduced the suspended matter 52 per cent., while detention periods of 9 to 24 hours in a septic tank effected a reduction of but 33 per cent. (*Jour. Assoc. Eng. Socs.*, 1906, vol. xxxvii, page 67.) This low percentage of removal was probably due to the iron wastes in the sewage. The ferrous sulphate was reduced to ferrous sulphide in such a fine condition that it escaped with the effluent and was recorded in the results of analyses as suspended matter.

The different nature of the sewage of different cities, differences in the period of sedimentation, and the tendency of the ebullition in septic tanks to increase the number of particles of solid matter in suspension in their effluents over the number in the effluents of sedimentation tanks make comparisons of sedimentation basin results at one place with septic tank results at another of little practical importance. At one plant, the suspended matter may have a large proportion of large, heavy particles, and at another plant the same total amount of suspended matter may

have a large proportion of very light, fine particles. The sedimentation of the former sewage will show a larger percentage removal than the detention of the latter sewage in a septic tank for the same number of hours. The difference will not be due to any advantage of sedimentation over septic action but to the variation in the settling solids.

TABLE 87.—COMPARATIVE RESULTS OF PLAIN SEDIMENTATION FOR 24 HOURS AND SEPTIC SEDIMENTATION FOR 21.6 HOURS

(Fifth Report, Royal Commission on Sewage Disposal, Appendix IV, pages 174 and 179; results in parts per million)

	Screened sewage	Sedimentation effluent		Septic effluent	
		Analysis	Reduction, per cent.	Analysis	Reduction, per cent.
Ammoniacal nitrogen	46.5	49.7	7	52.7	13[1]
Albuminoid nitrogen	10.0	8.4	16	8.0	20
Total organic nitrogen	19.2	16.9	20	17.4	18
Total nitrogen (Kjeldahl)	67.2	65.9	2	70.8	5[1]
Oxygen consumed at 27°C. at once.	23.8	19.7	17.2	19.9	16.4
Oxygen consumed at 27°C. in 4 hrs.	100.1	75.5	23	77.0	23
Solids in suspension	215.0	93.9	56	95.7	55

[1] Increase.

Inasmuch as the qualities desired in the effluent from a septic tank are chemical as well as physical, it is manifestly important to consider the possibility of the sewage reaching the works in different conditions[1] at different times. It has already been shown in Chapter V that if sewage remains too long in the presence of sludge undergoing anaerobic decomposition, it acquires properties which are undesirable from all viewpoints. Sewage having such properties is termed "over-septicized," and an installation of septic tanks should be so designed that a competent operator of the plant can prevent over-septicization.

In some cities industrial wastes are discharged into the sewers at certain hours and reach the treatment plant without much dilution. If there is ample tank capacity to receive and dilute these wastes, not only will they be unlikely to cause fluctuations in the results accomplished by the septic tanks, but they will also be of little significance

[1] An unusual instance of the effect of chemical composition on septic action was observed in the experiments of Kinnicutt and Eddy with a closed septic tank at Worcester. During 1901 they found that the average amount of free sulphuric acid in the sewage was about 100 parts per 1,000,000, but this did not disturb the septic conditions. During a short period when changes were being made in the sewerage system, the amount of acid increased to over 200 parts, which caused a great reduction in septic action, as shown by the absence of gas formation. (Fourth Report, Connecticut Sewerage Commission, page 27.)

during the stages of treatment carried on by contact beds or filters. The equalizing effect of ample storage facilities on the variable character of sewage reaching the works may be an important feature for the designer to consider.

Over-septicization.—Long before over-septicization had received a name its disadvantages were known. John W. Alvord called attention to them a number of times, as a result of his observations of the operation of early American septic tanks built from the plans of Alvord & Shields. In 1898 investigations were made at the Lawrence Experiment Station which indicated that sewage which has traveled for any considerable distance in sewers will, when it reaches its point of disposal, be practically free from dissolved oxygen, and need not remain in the septic tank longer than is necessary for a precipitation of the suspended organic matter and the accumulation of the fats upon the surface of the sewage. The report for 1899 modified this statement by saying that, to obtain the best results, sufficient time for the installation of the desired bacterial life is necessary, and this time varies with different kinds of sewage and at different seasons. The danger of keeping sewage too long in the tanks was explained as follows:

"It seems probable—although this is not yet proved—that the anaerobic action has been carried so far before the sewage reaches the filters that various bodies have been generated in the sewage which prevent the development of the nitrifying bacteria in the filters. The sewage has an odor more nearly resembling that of the wastes from a cesspool than that of ordinary fresh, stale or septic sewage. It is certain that, if the anaerobic process is carried too far, there may be a formation of distinctly poisonous bodies, which might prevent nitrification."

Further investigations showed (1900 report, page 481) that when a small volume of sewage is applied to a sand filter nitrification will take place, no matter to what degree of putrefaction this sewage has attained at the time of its application, if there is an abundance of air to come into contact with the sewage. When, however, sewage in an advanced state of putrefaction is applied to a contact filter and the entire open space of the filter filled with this sewage, Clark and Gage considered it possible that oxidation may be so rapid that the supply of oxygen within the filter will be exhausted before the process of nitrification has had time to begin. The slow absorption of oxygen by fresh sewage and its rapid absorption by very stale or septic sewage was proved by a number of experiments. The application of a small volume of over-septicized sewage to an intermittent sand filter, where there is an opportunity for considerable oxidation on the surface of the bed and there is a large amount of air in the filtering medium, presents very different conditions from those of a contact bed, having less opportunity for nitrification. In 1901 it was found that by aerating an over-septicized sewage it

could be treated satisfactorily in a contact bed, which was impossible without the aeration.

"As regards the question whether it is possible to over-septicize a sewage, several witnesses have stated that if sewage be kept too long in the tank, the amount of sulphureted hydrogen produced is considerably increased, and some witnesses have expressed the view that too long a stay tends to impair the subsequent oxidation of the sewage during its passage through the filters." (Royal Commission on Sewage Disposal, Fifth Report, page 28.)

Number of Tanks.—The size of tanks depends on the necessary detention period, character of sewage, space desired for sludge accumulations in the tanks, and local topography. The best size having been decided upon, the number of tanks depends upon the quantity of sewage to be treated. In some of the early American installations a single tank was adopted, but provision was made for placing movable partitions within it so as to form eventually basins of the size which experience indicated to be most suitable. It is now customary to construct several tanks, sometimes of the same size and sometimes of different sizes, even in the case of small treatment plants for institutions and small cities. An installation at Sturgis, Mich., in 1913, consists of a 24 × 36-ft. tank, one of 12 × 36 ft. and three of 12 × 12 ft.; this plant is for a city of 5000 population. For an asylum at Wernersville, Pa., the State Health Department installed four 8 × 84-ft. tanks; the treatment plant at this institution was designed to treat 300,000 gal. daily. Such subdivision of tank capacity not only gives to an intelligent operator the means of adjusting his plant to the fluctuations in flow, but also permits him to dilute trade wastes, avoid over-septicization, clean out his tanks in rotation so as to avoid great fluctuations in the quality of the effluent, and carry on experiments with different operating conditions.

An example of tanks arranged for both parallel and series operation is afforded by a plant constructed in 1908 at Washington, Pa., from the plans of R. Winthrop Pratt. These tanks, Fig. 92, are four in number, covered, each 25 × 100 ft. and designed to hold sewage between elevations giving an average depth of 8 to 10.5 ft. At the inlet end of the plant there is a gallery containing the end of a 14-in. force main through which the sewage is delivered. From this main, sewage is admitted to the upper end of each tank through two 12-in. submerged inlets (Fig. 93), and flows through the basin to three 12-in. submerged outlets at the other end. These outlets deliver the septic sewage into an outlet channel (Fig. 94), in which the height of the sewage fluctuates between limits controlled by automatic apparatus. If it is desired to employ series operation, stop planks are inserted in the outlet channel between tanks 3 and 4. Then tank 1 or tank 2, or both, as desired, will discharge

sewage through the outlet channel into tank 3. Here the direction of flow is reversed from that for parallel operation. On reaching the further end the sewage enters 3 inlets in an 18-in. vitrified pipe by-pass, which conveys it to the same number of outlets in tank 4. Here

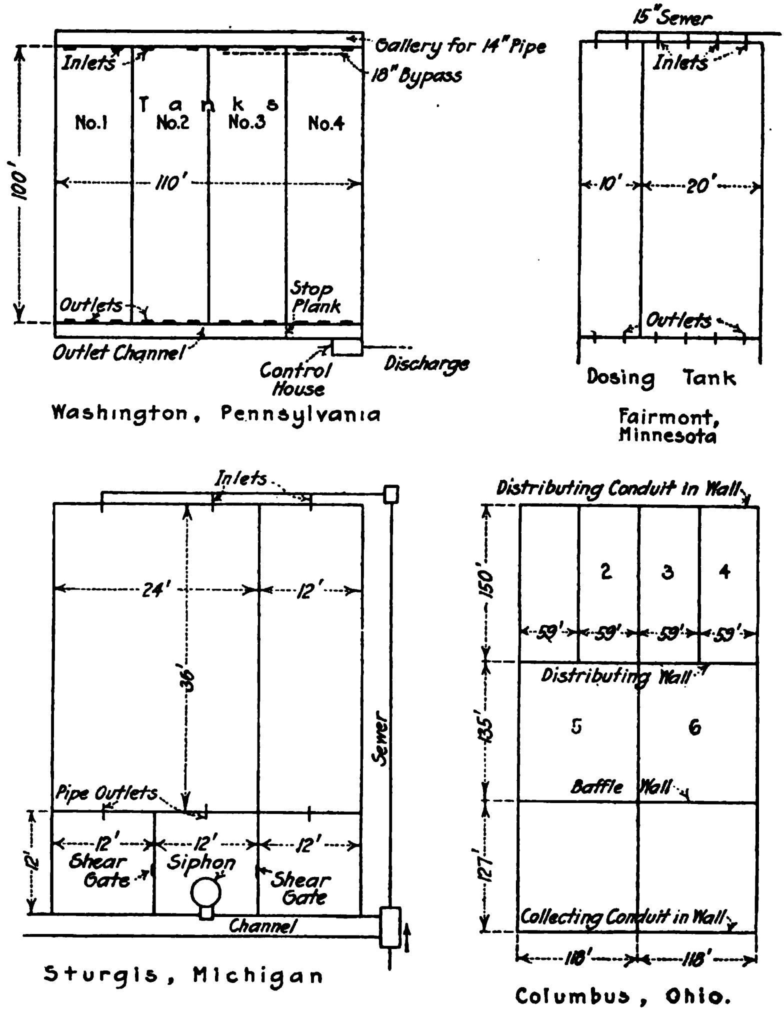


FIG. 92.—Different arrangements of septic tanks.

the normal direction of flow is maintained and the septic sewage passes into the end of the outlet channel.

The plan of the tanks at Fairmont, Minn., Fig. 92, shows an arrange-

ment adopted in a number of cases by Prof. Anson Marston. This example was for a place of about 3000 persons. The 2 tanks, which are of unequal size, operate in parallel, and have a combined capacity of 80,000 gal. The sewage is delivered from them to 2 intermittent sand filters.

An unusually flexible arrangement of tanks was selected by George S. Pierson for use at Sturgis, Mich. As described at the 1914 meeting of the Michigan Engineering Society, it consists of 5 compartments 8.5 ft. deep, arranged as shown in Fig. 92. By various settings of the gate valves on the inlets and of the shear gates, the installation may be given capacities of 31,600, 37,700, 46,600, 52,700, 58,800, 72,100 or 78,200 gal.

The large installation of septic tanks at Columbus, Ohio, is arranged as shown in Fig. 92. They were described in detail by their designer, John H. Gregory, in *Trans. Am. Soc. C. E.*, vol. lxvii, 1910, page 294. Basins 1 to 4 inclusive are called primary tanks, and basins 5 and 6 secondary tanks. They are approximately 12 ft. deep to the high-water line. The wall along the upper end of the primary tanks has a 5.5-ft. conduit within it, by which sewage can be turned into any tank through four 24-in. sluice gates. In the distributing wall at the lower end of these tanks is an overflow channel at the top, a 5-ft. collecting and distributing conduit in the center and a 5 × 3.3-ft. blow-off conduit at the bottom. The sewage passes from each tank into the collecting and distributing conduit through four 24-in. sluice gates, and passes from it through eight 24-in. sluice gates into either or both of the secondary tanks. The sewage is drawn from each of the latter basins through eight 24-in. sluice gates into a collecting conduit in the end wall. The conditions governing the general design were stated by Gregory as follows:

"The tanks were designed with the object of not only placing the outlets as far as possible from the inlets, but also of keeping the sludge as much as possible away from the outlets, this being accomplished by dividing the tanks longitudinally by transverse walls into 3 sections. To meet varying conditions in the character of the sewage and in the quantity treated, the design was made very flexible, it being possible to use any combination of one or more primary tanks with either or both secondary tanks. . . . . Each secondary tank is divided into 2 sections by a low baffle wall, to hold back the sludge at the bottom of the upper half of the tank, the top of the wall being about 1 ft. below low-water level. On the upstream side of the wall, and hinged to it, there is a floating scum board, of cypress, to hold back any scum which might form in the upper half of the tank."

The operation of this series of basins was originally intended to afford a practically constant rate of discharge through the 24 hours. With an average dry-weather flow of 20,000,000 gal. per day, the depth of the

sewage under such conditions would fluctuate between 8.9 and 12 ft., and the average period of detention would be about 8.5 hours. It was found more satisfactory, however, to maintain a constant level in the tanks. A method of operation found useful during the late spring, summer and early autumn was described as follows by C. B. Hoover:

"One secondary and one or two of the primaries are used continuously until septic action becomes violent enough to interfere seriously with the subsidence of suspended solids, then these tanks are put out of service and the other secondary and primaries are put in service. The tanks which are put out of service are allowed to stand idle until the deposited sludge has been somewhat well digested. When this condition has been attained, it will be indicated by the comparative absence of boiling, and a change in the color of the supernatant liquor from jet black to a clearer greenish color. The green color is due to a very luxuriant growth of Uglena, and this liquor has often been found to be supersaturated with dissolved oxygen, presumably as a result of the activities of these micro-organisms. When the tanks are shifted, the clear liquor in those which have been standing idle is drained into the river, and they are again ready for service, the sludge remaining in the tanks. The supernatant liquor which is drained into the river has been found non-putrescible and very low in bacteria (less than 10,000) and suspended matter."

Volume of Sludge.—In computing the operating capacity of the tanks, the space occupied by sludge must be estimated by the same methods followed in the case of a sedimentation basin and the result modified to allow for the dissolution of part of the solids. Inasmuch as this change takes place only in the organic matter, it may be desirable where combined sewage must be treated by anaerobic methods, to pass the sewage through grit chambers before it reaches the septic tanks, but there may be some cases where combined sewage will not readily part with the suspended mineral matter in a grit chamber unless the velocity is checked so that organic matter also settles out, forming an offensive mass troublesome to dispose of. If the grit is first removed, the amount of solids settling from combined sewage during dry weather in septic tanks will probably not be materially different in character from that obtained from separate sewage. The additional organic matter washed into combined sewers by storm run-off can be estimated only from local conditions, among which the efficiency of the street cleaning service is prominent.

The amount of sludge produced by septic tanks in a number of places is given in Table 88. The figures have been obtained from many sources, not always the original sources, and are chiefly of value as indicating the need of experience and good judgment in estimating such volumes without the help of analytical results.

The reduction in volume of sludge in septic tanks, for which the

TABLE 88.—QUANTITY OF SLUDGE PRODUCED BY SEPTIC TANKS

Location of plant	Velocity in chamber, ft. per hour	Detention period (hours)	Time elapsed since last cleaning	Water in sludge, per cent.	Sludge	
					Cu. yd. per mil. gal. sewage treated	Cu. ft. per capita per day
Accrington & Church, Eng.	7.25	42	1 year	90	7.7	0.006
Andover, Eng.	4.15	19.3	3 years	......	3.3	0.0033
Ashland, Ohio	17.70	1.5	9 months	......	0.65	0.0036
Birmingham, Eng.	19.20	6½	1 year	95	10.61	0.012
Boston, Mass. Exp.						
Tank 5	....	48–12	....	90	1.00[2]	....
7	....	12	....	90	0.64[2]	....
8	....	48	....	90	2.85[2]	....
9	....	24	....	90	1.56[2]	....
10	....	24	....	90	1.58[2]	....
Chicago, Ill. 39th St. Testing Sta.	....	8	10 months	89.4	3.1 max. 0.9 min.	....
Columbus, O. Exp.						
Tank A[1]	....	13.9	....	90	2.35[2]	....
B[1]	....	21.8	....	90	3.25[2]	....
C[1]	....	8	....	90	1.34[2]	....
D[1]	....	4	....	90	2.23[2]	....
E	....	8	....	90	4.80[2]	....
Depew, N. Y.	2.5	22	1 year	......	1.07	0.001
East Chicago	9.0	10	1 year	......	1.06	0.0023
Exeter, Eng. (small plant)	2.70	24	3 years	......	1.83	0.0023
Gloversville, N. Y., Exp.						
Summer	....	8	....	90	3.18[2]	....
Winter	....	6–10	....	90	9.20[2]	....
Weighted Ave.	....	....	....	......	6.35	......
Guilford, Eng.	3.05	23	1.6 years	......	4.4	0.0036
La Grange, Ill.	8.35–5.55	12–18	1 year	......	0.10	0.0002
Lakewood, Ohio	6.0	12	6 years	......	0.58	0.0011
Lawrence, Mass.						
Exp. Tank						
A 1898–04	....	42–14	....	90	1.60[2]	....
A 1904–06	....	12–36	....	90	1.13[2]	....
G	....	6	....	90	0.56[2]	....
H	....	18	....	90	0.94[2]	....
Manchester, Eng.	19.65	15.3	1 year	89	15.8	0.021
	21.25	14.1	1 year	88	13.6	0.019
	19.10	15.7	1 year	88	14.3	0.019
Mansfield, Ohio	4.2	24	4 years	80.8	0.82	0.0022
Pawtucket, R. I.	5.50	18.1	10 months	81.75	5.43	....
Plainfield, N. J.	14–35	9	1 year	89	3.05	0.007
Rochdale, Eng.	5.35	30	2.4 years	......	5.7	0.0021
Saratoga, N. Y.	9.15–6.05	10–15	2 years	87½	0.11	0.00017
Waterbury, Conn. Exp.						
Tank 2	0.95	15.5	....	86.3	1.07	....
3	1.29	11	....	85.4	0.55	....
Worcester, Mass., 1901	0.78	18	1 year	90	2.15	0.0046
Worcester, Mass., 1902	0.78	18	2 years	89	1.94	0.0025

[1] Sewage passed through grit chamber before entering septic tank.
[2] Computed for 90 per cent. water in sludge, and a weight of 1700 lb. per cubic yard.

designer must make some allowance according to a few authorities, has been variously estimated. It seems probable that it depends in a measure upon the character of the sludge and the extent to which the ebullition of gases lift particles of sludge into the sewage as it flows through a basin. The amount of suspended solids leaving a tank in the effluent is sometimes over 200 parts per 1,000,000, and generally is considerably in excess of the amount in the effluents from sedimentation and chemical precipitation basins, particularly after the septic tank has been in operation for some months. This is illustrated by experience with the septic tanks at Columbus, Ohio, where stale sewage is treated. C. B. Hoover, chemist in charge of the sewage treatment works, stated in his report for 1911 that two typical cycles of a tank after cleaning showed the increases in the mean total suspended matter in the effluent, as the period of time after a cleaning increased, which are given in Table 89. The sludge that remains on the bottom becomes more dense the longer it stays in the tank and some of the loss of volume formerly attributed solely to gasification and liquefaction is now considered to be due to this purely mechanical process.

Table 89.—Total Suspended Solids in Septic Tank Effluents, Columbus

(Parts per million)

Period in service	First cycle			Second cycle		
	Sewage	Effluent	Reduction, per cent.	Sewage	Effluent	Reduction, per cent.
First week	241	94	61	312	88	71
Second week	208	94	55	317	88	72
Third week	312	115	63	262	116	56
Fourth week	255	106	58	294	147	50
Fifth week	253	143	43	.....	.....	....
Sixth week	212	144	32	.....	.....	....

These results were obtained in June, July and August.

There are two 50 × 100-ft. and two 50 × 200-ft. tanks at Plainfield, N. J. The long tanks have a baffle 4 ft. high across their middle. They receive screened sewage, mainly domestic. With clean tanks, the period of detention in both long and short tanks being 9 hours, the velocities would be 11.2 and 22.4 ft. per hour. With sludge in the tanks, which have a total effective depth of 6 ft., the short basins were found to have detention periods of 6.5 to 7.5 hours, corresponding to velocities of 15.4 to 13.3 ft. per hour, and the long tanks had detention

periods of 5.6 to 6.7 hours, corresponding to velocities of 35.7 to 29.8 ft. per hour. The tanks were cleaned in March of each year; their efficiency is shown in Table 90.

TABLE 90.—REMOVAL OF SUSPENDED SOLIDS AND ORGANIC MATTER, BY SEPTIC TANKS, PLAINFIELD, N. J., 1909–11

(Parts per million. Roy S. Lanphear, *Engineering Record*, January 13, 1912)

Month	Suspended matter									Organic matter (oxygen consumed)								
	1909			1910			1911			1909			1010			1911		
	Screened sewage	Septic effluent	Percentage reduction	Screened sewage	Septic effluent	Percentage reduction	Screened sewage	Septic effluent	Percentage reduction	Screened sewage	Septic effluent	Percentage reduction	Screened sewage	Septic effluent	Percentage reduction	Screened sewage	Septic effluent	Percentage reduction
Jan....	110	48	56.0	116	50	57	167	73	56.0	89	55	38.0	74	56	24.0	92	64	30.0
Feb....	104	56	46.0	114	50	56	141	76	46.0	87	53	39.0	74	53	28.0	85	63	26.0
March[1]	102	58	43.0	133	58	56	155	56	64.0	81	54	33.0	78	56	28.0	83	61	26.0
April..	122	46	62.0	138	45	68	177	55	69.0	76	46	40.0	71	50	30.0	91	59	35.0
May...	164	39	76.0	168	42	75	221	59	73.0	75	45	40.0	74	52	24.0	85	57	33.0
June...	126	57	55.0	156	47	70	221	81	63.0	64	47	26.0	72	51	29.0	85	57	33.0
July...	126	67	47.0	142	65	54	184	61	67.0	58	48	17.0	62	49	21.0	69	53	23.0
August	166	55	67.0	146	55	62	178	55	69.0	62	45	27.0	60	46	23.0	77	50	35.0
Sept...	132	39	70.0	152	55	64	200	55	72.0	66	45	32.0	72	50	31.0	81	51	37.0
Oct....	145	50	66.0	196	60	69	185	67	64.0	67	49	27.0	86	55	36.0	81	57	30.0
Nov....	179	57	68.0	271	69	74	...	...	....	84	57	32.0	100	60	40.0	...	...	....
Dec....	135	57	58.0	191	72	63	...	...	....	82	58	29.0	92	60	35.0	...	...	....
Av.....	134	52	59.5	152	56	64	173	64	64.3	74	50	31.7	76	52	29.1	83	57	30.8

[1] Tanks usually cleaned during March.

The removal of suspended matter from the liquid passing through the tanks is about the same as at Columbus. Part of it remained in the tank as scum and part as sludge. The average specific gravity of the sludge was found to be 1.037 and the amount of water 89.4 per cent. The total volume of sewage treated during the test was 550,400,000 gal. The dry solids in the screened sewage amounted to 0.71 ton per 1,000,000 gal., based on influent and effluent data, and the dry solids in the effluent to 0.25 ton, so that 0.46 ton was deposited in the tank. The amount actually found in the scum and sludge was 0.28 ton, so that about 39.1 per cent. of the solid matter had been liquefied or gasified. Stagnation of sewage in the tanks for periods of 28 to 55 days showed a reduction in the volume of sludge by consolidation and liquefaction of 11.9 to 26.6

per cent. The reduction in organic matter by the Plainfield septic tanks is also given in Table 90, and the removal of bacteria in Table 91.

TABLE 91.—REMOVAL OF BACTERIA BY SEPTIC TANKS, PLAINFIELD, N. J., 1909–11

(Counts in millions. Roy S. Lanphear, *Engineering Record*, January 13, 1912)

Month	1909			1910			1911		
	Screened sewage	Septic effluent	Percentage removed	Screened sewage	Septic effluent	Percentage removed	Screened sewage	Septic effluent	Percentage removed
Jan.	2.49	1.09	56	......	......	......	2.00	1.06	53
Feb.	1.89	0.87	54	1.27	0.95	25	1.61	0.99	39
Mar.	2.02	0.92	54	2.04	2.44	......	1.72	1.76	......
April	2.43	1.63	33	2.27	1.43	37	2.23	1.97	12
May	2.22	1.16	48	2.66	1.44	46	3.24	2.10	35
June	2.21	0.97	56	2.87	1.40	51	3.71	2.00	46
July	1.92	1.24	35	2.67	1.26	53	3.11	1.88	40
Aug.	2.09	0.98	53	2.96	1.69	43	3.31	2.22	33
Sept.	2.47	1.03	58	2.59	1.43	45	3.26	2.54	22
Oct.	2.47	0.86	65	2.22	1.09	51	2.61	2.08	20
Nov.	2.13	0.75	65	2.37	1.57	34	......	......	......
Dec.	2.43	0.81	67	2.09	1.25	40	......	......	......
Average	2.23	1.03	54	2.36	1.45	39	2.68	1.86	31

The septic tank installation at Columbus, Ohio, consists of 2 large and 4 small basins (Fig. 92), any two or more of which may be used together. The sewage is screened, passed through grit chambers, and pumped to the tanks. They have a combined capacity of 8,000,000 gal. The depth is 12 ft. Their record during the first years of their operation is as follows:

Year	1909	1910	1911	1912	1913
Detention period, hours	10.1	7.1	6.2	4.9	4.0
Suspended solids removed, per cent	58.0	62.0	65.0	55.5	60.0
Dissolved oxygen cons. reduced, per cent.	......	28.0	27.0	26.0	29.0

These figures agree reasonably well with British results, such as those given in Table 92.

TABLE 92.—EFFECT OF DIFFERENT DETENTION PERIODS IN OPEN, RECTANGULAR SEPTIC TANKS, AT LEEDS, ENGLAND

(Report on Experiments on Sewage Disposal, Leeds, 1905, page 66)

	Parts per mil. in eff.	Reduction, per cent.	Parts per mil. in eff.	Reduction, per cent.	Parts per mil. in eff.	Reduction, per cent.	Parts per mil. in eff.	Reduction, per cent.
Detention period, hours	12		24		48		72	
Total solids......	1253	......	1117	......	1123	......	1055	......
Suspended solids..	274	52	163	71	155	73	141	76
Free ammonia....	22.3	22	21.5	24	23.1	19	25.5	37
Albuminoid ammonia.	7.7	50	6.4	58	5.4	64	4.8	52
Oxygen consumed, 4 hours at 80°F.	74.4	45	69.0	49	61.4	55	51.3	55

The available information indicates that careful operation of a septic tank installation, capable of flexibility in capacity and detention period, will result in a reduction of about one-third in the weight of the solids deposited from average screened stale American sewage.

"At Huddersfield, Mr. Campbell estimated that about 38 per cent. of the solids were converted into gas or digested; at Leeds, Mr. Harrison put the figure at about 30 per cent.; at Manchester, Dr. Fowler says that possibly, though not certainly, about 25 per cent. of the total suspended matter is digested or converted into gas; at Sheffield, Mr. Haworth puts the digestion at 32.9 per cent., while at Birmingham, Messrs. Watson and O'Shaughnessy say that the figures available indicated a digestion of not more than 10 per cent. of the suspended matter entering the tanks." (Fifth Report Royal Commission on Sewage Disposal, page 21.)

The liquefaction of solids in a closed septic tank at Worcester, Mass., was estimated in the following manner by Kinnicutt and Eddy in the fourth report of the Connecticut Sewerage Commission, page 39. During a period of 2 years 3 months, the difference between the results of analyses of the influent and effluent showed that 1194 lb. of suspended matter had been held back in the tank. The sludge at the end of this period contained 729 lb. of dry solids, indicating that 465 lb. had been liquefied. This was a reduction of nearly 39 per cent. The liquefaction during the first 15 months was 28 per cent., and during the last year nearly 48 per cent.

Such experience indicates that provision must be made for storing about 60 or 70 per cent. of the suspended matter not removed by coarse screens and grit chambers, plus its accompanying water. The total amount of storage to be provided will be fixed by the assumed lapse of

time between cleanings of tanks. This will depend in turn on the condition of the sewage reaching the works and the method of disposal of the tank effluent. During very warm weather the Columbus sewage sometimes reaches the tanks in such a septic condition that the basins must be operated as sedimentation tanks solely, in order to prevent advanced septic action, and the discharge with the effluent of so much suspended matter that the trickling filters receiving it will be materially affected. On the other hand, several weeks must sometimes elapse before septic action begins in a tank started with clean bottom and walls, unless it is filled at the outset with septic sewage from other tanks.

There is always the probability that septic tanks built for small American towns will not receive regular attention from a competent man, and the small basins are usually given large sludge-storage capacity in consequence. In designing such an installation there are two alternative risks to be faced, over-septicization of the sewage due to a protracted stay in the tank, or large amounts of suspended matter in the effluent caused by high velocities of flow. With the fresh weak sewage likely to be received at such small plants, the latter trouble is more probable than the former, and consequently it will be well to provide storage for at least 9 months' accumulation of sludge consisting of 60 per cent. of the total suspended matter and enough water to render the latter about 90 per cent. of the total volume. Liquefaction of the solids may reasonably be expected to keep tanks designed on this basis in fair working condition for a somewhat longer period. With plants for larger cities, where fairly competent supervision is likely to be given, the provision for sludge storage may be reduced by designing the tanks so that a small amount of the oldest part of the sludge is drawn off at frequent intervals. This procedure is much favored in England, and the bottom slopes, sludge channels and sludge gates are designed to facilitate it. With frequent removal of the sludge, the percentage of water in it will be higher than when a long period of digestion is permitted, and the designer must keep in mind the fact that sludge with 95 per cent. water occupies twice the space required to store sludge consisting of the same amount of solids, but with only 90 per cent. of its volume composed of water.

"Our experience points to the conclusion that there can be no definite rules as to how long a septic tank should be run without cleaning, and that it must be left to the manager of each works to see that the suspended solids issuing from the tank do not increase to such an extent as to have an injurious effect upon the filters or upon the quality of the filtrate. The point at which injury to the filters or deterioration of the filtrate will occur will largely depend upon the nature and strength of the sewage, the kind of filter used, the size of the material in the filter, and the rate of filtration. At Accrington, for instance, something like 15 parts (per 100,000 parts)

of suspended matter in the septic tank liquor can be put upon the coarse percolating filters, without any danger of clogging, whereas at Manchester and Burnley, this would be too much for the contact beds. Again the successful working of some forms of fine percolating filter (*e.g.*, that at Friern Barnet), depends upon the time allowed for the drying of the sludge on the surface of the material between two applications of tank liquor. A tank liquor containing, say, 10 parts of suspended matter per 100,000, might be conveniently treated on a filter of this kind, if the delivery were made once every 3 days; but if it were made once every day, the sludge caught on the surface of the filter would probably not have dried sufficiently to allow of its being scraped off before the next delivery was made." (Fifth Report, Royal Commission on Sewage Disposal, page 24.)

Reduction of Dissolved Organic Matter.—In addition to changing the character of the sewage by the removal of part of its suspended matter, a well-operated septic tank usually makes a reduction in the dissolved organic matter in the raw sewage. There has been a great difference of opinion regarding the amount of this decrease. Dr. W. P. Dunbar states that at Hamburg the reduction in oxygen absorbed was about 33 per cent., in albuminoid ammonia about 23 per cent., in organic nitrogen about 37 per cent. and in organic carbon about 40 per cent. ("Sewage Treatment," page 86.)

Clark and Gage reported, as a result of investigations at Lawrence lasting many years, that the reactions taking place in different septic tanks may be different when every effort has been made to operate them in parallel, and for this reason it is impossible to estimate closely the effect of the rate of flow through a septic tank upon its efficiency as a preliminary treatment for sewage. (Report Mass. St. Bd. Health, 1908, page 479).

Roofs.—The early opinion that septic tanks must be covered to enable digestion of the solids to proceed satisfactorily is no longer held by engineers. Many tanks, particularly those of small plants, are provided with roofs for other reasons, among which the diminution of odors from the works is often prominent. In the anaerobic decomposition of sludge, the gases evolved are sometimes extremely offensive. By preventing any agitation by the wind of the contents of septic tanks, the odor from them is rarely so strong as to cause annoyance beyond the immediate vicinity of the basins. A powerful stench is often produced when septic sewage is sprayed over trickling filters, and where such filters are employed the usefulness of roofs is mainly to keep the sewage warm and out of the sight of the public and, in small plants without attendants constantly on duty, to keep off children and irresponsible persons.

Occasionally the roofs are constructed of wood, but more often are reinforced concrete slabs carried by reinforced concrete beams. The roofs of the septic tanks at Saratoga Springs are groined arches. The

sewage is somewhat warmer than the outside air during a part of the year and the inner surfaces of the walls and roof are liable to become covered with moisture. This water may absorb the hydrogen sulphide given off by the decomposing sludge, causing deterioration of the concrete. A noteworthy instance of this occurred at Hampton, England, where concrete surfaces above the sewage level became much softened and large flakes could be detached by the fingers. Some American experience of this nature is reported in *Engineering Record*, May 16, 1908, and *Engineering News*, December 12, 1912. All surfaces liable to exposure to such action should be made as dense and smooth as practicable, not only to prevent external deterioration but also the penetration of acid moisture to the steel reinforcement.

Where roofs are employed, the tanks must have vent pipes in the roofs in order that the rapid entrance or escape of a large amount of sewage shall not cause damage on account of the difference in air pressure above and below the roof. As the gases[1] which escape from a septic tank are sometimes explosive when ignited, notice to this effect should be posted near all entrances to roofed tanks.

One of the earliest tank installations in the United States, that at Saratoga Springs, N. Y., was considerably injured by an explosion on January 26, 1906, which lifted the entire roof of a 51.5 × 91.5-ft. basin, completely ruining it. The roof had 6 manholes closed with iron covers. The opinion of F. A. Barbour, designer of the plant, and of some other engineers who discussed the accident at the time, was that the explosion was caused by a burning match or glowing ashes being dropped through a manhole by one of the workmen about the tanks.

An explosion took place in a covered septic tank at Florence, N. C. (*Eng. News*, Feb. 25, 1915), in which a sludge-ripening process was proceeding. The tank had been thoroughly cleaned out some time previously. The odor of sulphureted hydrogen was very distinct. The attendant set fire to dead weeds and grass in order to clean up around the plant, and the flames evidently ignited gas escaping through a large crack in the roof of the tank, for a heavy explosion occurred inside the tank followed by the escape of heavy white smoke from manholes with loose covers.

Explosions in covered septic tanks have been reported from Cromer, Exeter, Ilford and Sheringham, England.

[1] An analyasis of the gas from an air-tight septic tank was recorded as follows by the Massachusetts State Board of Health in its 1899 report (page 422): carbonic acid, 3.4 per cent.; heavy hydrocarbons, 0.3 per cent.; oxygen, 0.5 per cent.; carbonic oxide, 0.6 per cent.; methane, 78.9 per cent.; nitrogen, 16.3 per cent. Kinnicutt and Eddy state that the average results of weekly analyses of the gas from a closed septic tank at Worcester gave 5.90 per cent. of carbon dioxide, 0.76 per cent. of oxygen, 75.18 per cent. of methane, and 17.40 per cent. of nitrogen. The amount of gas averaged 2.4 gal. per 100 gal. of sewage in 1901 and 3.9 gal. in 1902. About twice as much gas was evolved in warm as in cold weather.

Inlets and Outlets.—In the earliest designs for septic tanks, when it was believed that the best results could be obtained only by enclosed tanks, the sewage entered and left the basins at points 18 in. or more below the surface. This was done mainly to trap both openings and prevent the entrance of air into the tanks. It was not long before this reason for submerged outlets was abandoned, but other reasons for giving them such a location took its place. Scum on the surface of the sewage made it desirable to provide submerged inlet and outlets, or their equivalent. Another reason for submerged openings is that the action of the sewage tank, in liquefying organic matter, is apparently weaker at mid-depth than at the level of the sludge or just below the scum, and it is therefore desirable to admit and draw off the sewage near mid-depth. Among the experiments leading to this conclusion are some made at Plainfield, N. J., by Lanphear, reported in *Engineering Record*, January 13, 1912.

Although submerged openings are preferred by some engineers for the reasons stated, weirs are also employed in many septic tanks, but they are usually guarded by scum-boards from 2 to 4 in. in front of them, extending to a depth of at least 2 ft. The narrow openings between the boards and the weirs are easily kept free from scum, and the sewage is compelled by the depth of the scum boards to take about the same course it would follow with submerged openings.

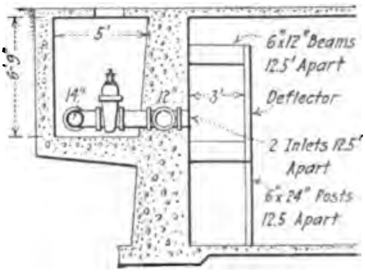


FIG. 93.—Inlet of septic tank, Washington, Pa.

An example of pipe-and-branch inlets is afforded by the septic tanks at Washington, Pa., designed by Pratt and described by him in *Engineering News*, July 16, 1908. In this arrangement (Fig. 93) the sewage reaches the plant in a 14-in. force main carried in a pipe gallery along the end of the tanks. Opposite the center of each tank there is a 12-in. tee with a gate, connecting with a pipe of the same diameter in the wall. This pipe conveys the sewage to two submerged openings 12 in. in diameter and 12.5 ft. apart. Three feet in front of these openings there is a reinforced concrete deflector extending the whole width of the tank.

In many cases the pipe inlet or outlet is not built into the wall but is held a few inches from it. This enables the bottom to end in a tee, with two of its openings discharging the sewage and the branch opening connected to the riser. Such an arrangement is shown in Fig. 102.

In the septic tanks at Mansfield, Ohio, F. A. Barbour employed at the end of each tank a 12-in. vitrified pipe with 4 branches passing

through the wall into the tank 2.5 ft. below the level of the sewage. Each of these headers is supplied by a branch from a cast-iron supply pipe reducing from 24 to 18 in. in diameter after passing the end of two of the tanks.

A submerged outlet provided with a shear gate is shown in Fig. 94. This design was used by Pratt at Washington, Pa. There are three of these outlets at the end of each tank 25 ft. wide. In an article in *Engineering News*, July 16, 1908, Pratt states that a baffle wall is placed in front of the outlets.

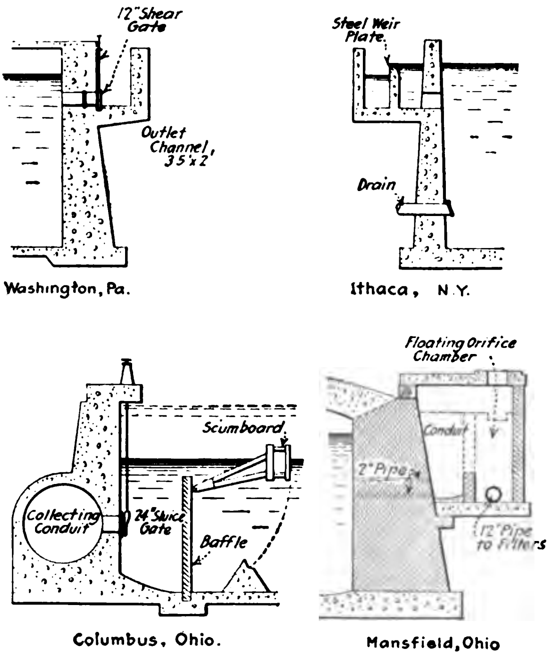


Fig. 94.—Outlets of septic tanks.

A submerged outlet combined with a weir, used by Ogden in the septic tanks at Ithaca, N. Y., is shown in Fig. 94, from *Engineering Record*, Feb. 1, 1908. Each of the tanks has at one end 2 openings, 12 in. high and 24 in. long, through which sewage passes into an open channel from the tank. This channel is divided by a longitudinal wall with a weir plate along its crest. This plate is 4 × ½ in. in section, and the sewage drops over it as shown in the illustration into the outer channel, which has a transverse weir at its discharge end, with a crest about 14½ in. below that of the other.

The form of outlet used at Columbus, Ohio, in 1904, by Gregory is

also shown in Fig. 94, from *Trans. Am. Soc. C. E.*, vol. lxvii, 1910, page 290. The wall is a combined section, consisting of a concrete conduit 5.5. ft. in diameter surmounted by a low retaining wall. The sewage is drawn off through 24-in. sluice gates. There is a thin reinforced concrete baffle surmounted by a hinged scum board to guard the openings.

The outlet employed by Barbour for four 92.25 × 52-ft. septic tanks at Mansfield, Ohio, is unlike the others shown in Fig. 94. At the outlet end of each tank there are two horizontal rows of 2-in. pipe running through the wall. The rows are 6 in. apart vertically, and the pipes in each row are 12 in. apart. There are 98 of these small outlets for each tank, and the upper row is 2 ft. below the surface of the sewage. They discharge into a concrete conduit running along the outside of the end wall, 4 ft. deep, 2 ft. wide at the top and 18 in. wide at the bottom. Opposite the center of each tank this conduit passes through a chamber where the sewage is removed through a floating orifice. Each orifice is 2 × 12 in. and supported by an iron frame held in place by two 21 × 36 × 8-in. floats. The orifices can be adjusted for submersion to any depth down to 1.5 ft. The sewage which passes through the orifices is conducted by a pipe ranging from 10 to 15 in. in diameter to an aerating chamber.

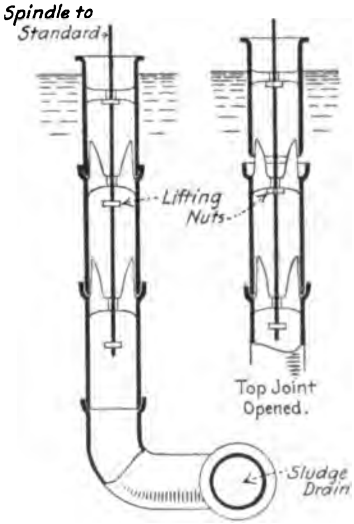


Fig. 95.—Draw-off pipe for septic tanks.

It is undesirable to use the floating arm (Fig. 84) for an outlet from septic tanks unless the lip is provided with a scum board, and where the scum is tough or thick some engineers consider it undesirable in any form. Hugh P. Raikes recommends ("Sewage Disposal Works") the substitute shown in Fig. 95. This is a vertical standpipe consisting of sections about 1 ft. long. Inside each section is a cross-arm with a hole in the center, through which passes a spindle raised and lowered by a gate-standard of the rising-stem type. Lifting nuts are fixed on the spindle at such points that the top joint must be opened before the one next to it, and both the first and the second joints must be opened before the third. By this detail, there is a minimum disturbance of the scum and sludge. In case it is desired to operate with a very thick scum the first and second lifting nuts can be adjusted so that the top joint will not be opened.

Bottom Slopes.—The slopes of the bottoms of septic tanks ordinarily do not differ from those employed with sedimentation basins. Septic sludge free from grit can be moved with scrapers on slopes of 1.5 to 3 per cent. without difficulty, but heavy scum, which settles on top of the sludge when the tank is drained, must be broken up with shovels or other implements, before it can be handled easily on such slopes.

Prof. Marston adopted much steeper slopes than 3 per cent. from the side walls toward the center of a basin. Where he employs three parallel tanks, a transverse section of them shows rather shallow side walls, deep longitudinal dividing walls, a central basin with its bottom sloping gradually toward a central sludge drain, and side basins with a steep bottom slope from the side wall and a moderate slope from the division wall toward a central sludge drain. John M. Farley also used steep bottom slopes, from 1:12 to 1:24, in his plans for the Mt. Vernon, N. Y., treatment plant. In the Baltimore tanks slopes of 1 on 4 were used by Calvin W. Hendrick for the floors of primary tanks and 1 on 12.19 in the secondary tanks. In Great Britain, a slope of about 1 on 15 is considered desirable for the discharge of sludge by gravity. Where scum forms and has to be removed with the sludge, greater bottom slopes are advisable than those where sludge alone must be removed.

The general arrangement of the bottom will usually be influenced to a considerable extent by the method of disposing of the sludge. Wherever practicable it should be discharged by gravity into a well from which it can be pumped or into a sludge drain. There is very little information regarding the resistance of pipe surfaces to the flow of sludge, but a helpful basis for design is afforded by the fact that no difficulty was experienced for several years in draining the Mt. Vernon septic tanks through a 16-in. pipe on a 1 per cent. grade. At this place each tank bottom is of the half-hopper type, with the sludge outlet in one of the side walls, so that the floor slopes to it from both end walls and the other side wall. The tanks are side by side, with bottoms at successively slightly lower elevations, enabling the sludge drain to be run transversely under each of the basins, without any sacrifice of grade.

Some engineers prefer to have the sludge outlets away from the sewage outlets, and in such cases the bottom usually slopes from each end toward a central transverse drain. The object of this design is to free the escaping liquid, so far as practicable, of particles of sludge carried up by gases. It is believed by those advocating the arrangement that ebullition is greatest where the depth of sludge is greatest, so that the low point of the floor should not be near the outlets. Other engineers, particularly in Great Britain, have stated that where large quantities of scum are anticipated, it may be desirable to slope the bottom toward the outlet end. This opinion rests on the belief that the scum will probably be thicker near that end than elsewhere, and it is wise to have

the scum dropped, in cleaning operations, as close to the sludge drainage openings as possible, where there is most space to receive it.

The sludge openings are usually closed by shear gates, unless they are so large that sluice gates are necessary for ready operation. All conduits and pipes for sludge should be run on straight lines between manholes, if possible, in order that any stoppage can be removed without serious difficulty.

Small septic tanks which collect only a small quantity of sludge and, on account of local conditions, could be drained by gravity only by the construction of expensive accessory works, can be cleaned by hand. The supernatant sewage which cannot be drained off by gravity and sludge of ordinary density can be pumped by diaphragm pumps into carts, as nightsoil is handled. This method of cleaning causes odor in the vicinity of the tank, but it is preferable in the case of some small installations to expensive drains in rock trenches or to far distant sludge beds. The floors of such small tanks may be left flat if the cost of excavation to give easy slopes to one point would be heavy.

THE TRAVIS TANK

The development of the Travis tank was outlined on page 20. Its features are shown in more detail in Fig. 96 from drawings furnished by Arthur E. Collins, City Engineer of Norwich, England, who designed the plant in 1909 with the assistance of Dr. Travis. There are 2 detritus tanks, 4 main or hydrolytic tanks and 4 finishing or hydrolyzing tanks in the installation, which has a rated capacity of 3,600,000 U. S. gal.

The sewage is pumped to a small chamber at the end of the force main, from which a channel leads to the end of each detritus tank. The inlets to the detritus tank, not shown in Fig. 96, are near each side of the tank and between them is a scum channel draining to a central sludge sump. The cross-section of the detritus tank is similar to that of the hydrolytic tank.

The sewage escapes from the 2 detritus tanks into a main inlet channel, from each end of which a branch runs to a secondary inlet channel across the end of two of the main tanks. From this channel the sewage passes through sluice gates and down drops to submerged inlets in the end wall of each tank near the side walls. The scum which gathers in the main tanks is removed through a channel between the 2 drop inlets.

The inlets admit the sewage to the two sedimentation chambers of each tank, in which are hung colloiders of hard wood, 1.5 × 0.75 in. in section and spaced 3 in. apart transversely and 5 to 9 in. longitudinally. Their purpose is to attract the fine non-settling solids and to produce de-solution of part of the colloids, as explained in Chapter VI, page

219. The portion of the sedimentation chamber near the inlets in which there are no colloiders and the similar portion near the outlet end are "devoted solely to the sedimentation of the heavier suspended and coagulated matter respectively," according to Collins. The solids pass through semicircular inlets into the central reduction chamber, being helped along by a slow flow of sewage through the inlets. At the end of each sedimentation chamber there is a weir over which the sewage passes to an effluent channel, leading around one side of the hydrolyzing chamber to the main effluent channel.

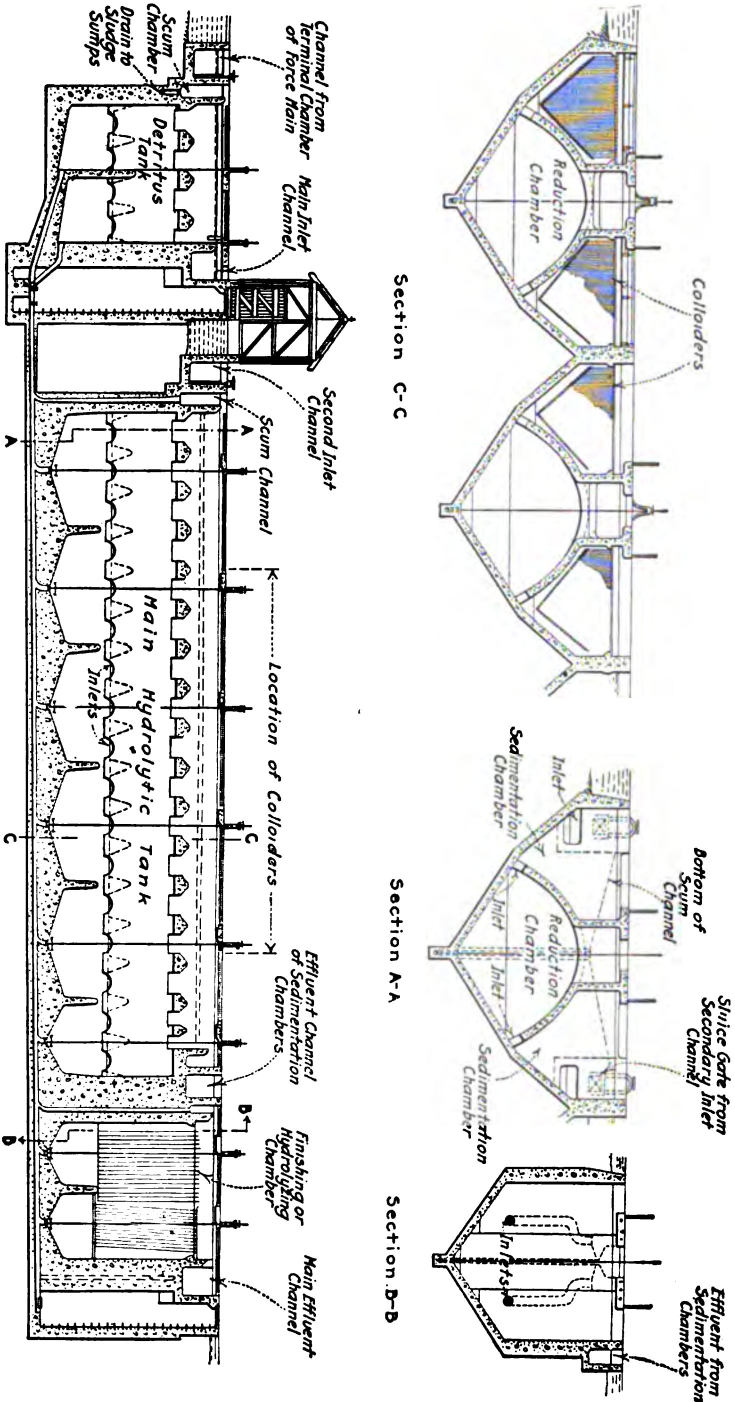
The sewage which passes down into the reduction chamber through the inlets from the two sedimentation chambers above it becomes septic and the sludge in the hopper bottoms of the reduction chamber is drawn off through a system of drains to a sludge sump. At the end of the chamber the sewage passes over a weir to drop shafts leading to submerged inlets in the hydrolyzing chamber. This chamber has a scum channel at the inlet end and an outlet weir at the other end, which is crossed by the main effluent channel. During the summer as much as 3 in. of scum sometimes floats in the central compartments, but little is observed in the side compartments.

The arrangement of the colloiders has been modified since the works were put in operation. They do not sludge up but rid themselves of sludge. There is little or no ebullition, as the sludge is removed frequently, from once a day to twice a week, and septic conditions are prevented as much as practicable.

The operation of the plant is controlled by the dimensions of the weirs over which the sewage leaves the 3 chambers of the hydrolytic tank. These dimensions are such that 40 per cent. of the sewage passes through the entire length of each sedimentation chamber, the detention period being about 4 hours, and the remaining 20 per cent. passes through the openings in the bottom of the sedimentation chamber and through the reduction chamber to the end weir, requiring about 12 hours for the passage.

The hydrolyzing chamber is provided to collect part of the large amount of suspended and colloidal matter which is contained in septic-tank effluents, particularly when the ebullition of gas is active.

The authors were informed by Collins in 1915 that compressed air is employed in cleaning the tanks. A small amount of suspended matter adheres to the sloping sides of the hydrolyzing chamber and is most easily removed with an air hose having a nozzle suited for the purpose. The sloping sides and bottoms of any extensions or new works should have a much steeper inclination, in his opinion, than in the first Norwich installation. The flat slopes make it necessary to remove the sludge with 92 to 95 per cent. of water.



Longitudinal Section Through Center of Reduction Chamber

FIG. 96.—Sections of the Travis tanks at Norwich, England.

THE IMHOFF OR EMSCHER TANK

The general construction of the Imhoff tank was described on page 216 and the theory of its action on page 217. The two main features of difference between the Imhoff and Travis tanks are that the liquid in the sludge chamber of the former is stagnant except as stirred by gases rising through it, and that the openings through which the solids pass into the sludge chamber are guarded by projecting lips below them so that no gases of decomposition can pass up through the slot into the sewage in the sedimentation chamber. This latter feature of design was also developed in the United States in February, 1907, by W. S. Shields, whose plans were officially submitted to his client on April 6 of that year. The plant was built in the autumn of 1907 and remained in continuous operation for some years with such success that Shields built several more upon the same principle. The Imhoff process patent[1] in the United States was filed on May 6, 1907.

There are two arrangements of Imhoff tanks, the radial flow and the horizontal flow. An example of the former is furnished by the new sedimentation plant at Baltimore designed under the direction of Calvin W. Hendrick by Leslie C. Frank. There are 28 Imhoff tanks in this plant, each of the type shown in Fig. 97, which was adopted as being less expensive than the rectangular type more often used. The size of the units was selected to secure flexibility in operation in connection with the large sedimentation basins of the original treatment works. Frank pointed out in *Engineering Record*, July 4, 1914, that such small individual units have a much shorter flow line than elongated multiple units, and if the flow line is not made too short this works to advantage because, with a given detention period, the velocity of flow is correspondingly low and hence it takes a higher unexpected overloading to reach the critical swirling velocity at which sedimentation is hindered.

The sewage passes from the main distributing channel through a trough 12 in. wide and 12 in. deep to the central distributing ring of the same cross-section. There are eight 12 × 12-in. openings in its bottom through which the sewage passes down into the sedimentation chamber; provision has been made for partially closing the openings near the inlet trough with flat plates if this seems necessary to secure an equal flow of the sewage. One purpose in admitting sewage to the sedimentation chamber in this way was to give it a downward velocity so that short-path surface currents would not exist. Deep baffle plates have also been used in radial flow tanks to attain the same end.

[1] The royalty charged for the use of the Imhoff tank by the Pacific Flush Tank Co., American representatives of Dr. Imhoff, is based upon the maximum population for which a plant is designed. For an ultimate population of 1000 the royalty is about $80; 5000, about $255; 10,000, about $400; 100,000, about $2500.

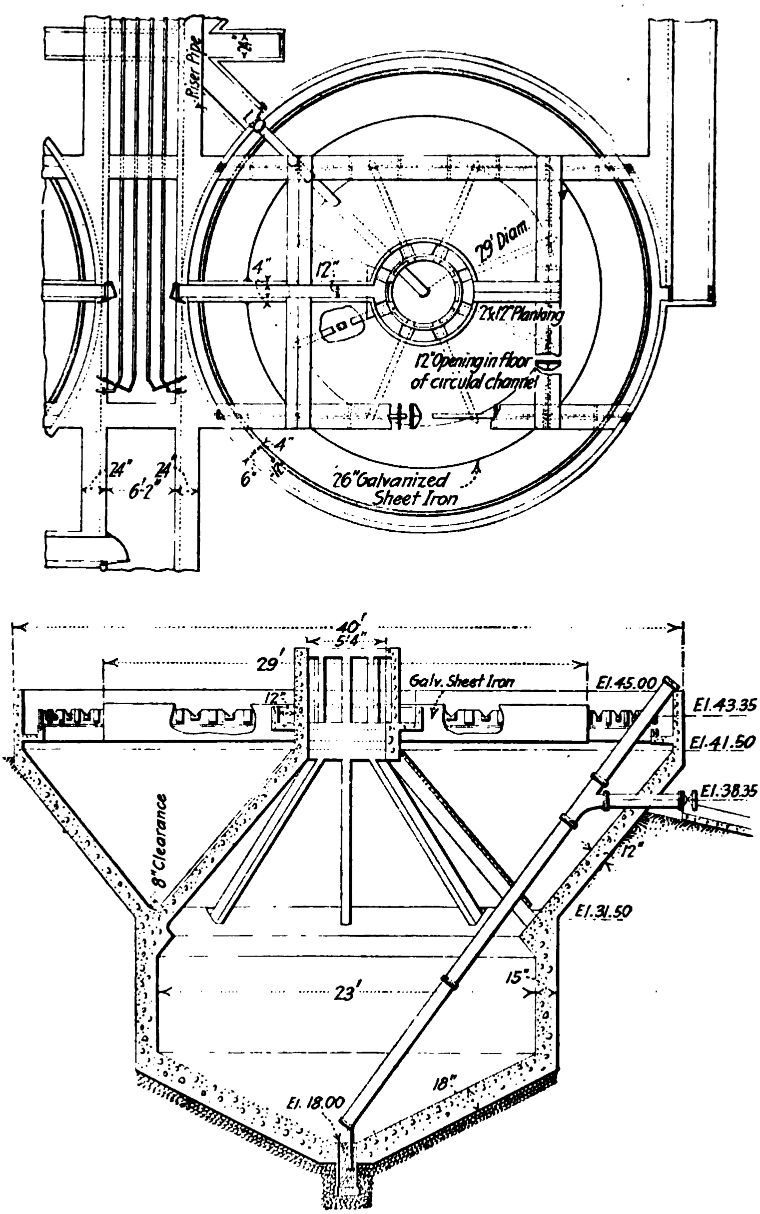


Fig. 97.—Radial-flow type of Imhoff tank, Baltimore.

The settled sewage is drawn off through 50 V-shaped weirs on the inner side of a circumferential effluent collector. The weirs are made of No. 14 galvanized iron plates, slotted so that they can be accurately adjusted on the cypress side of the channel by means of thumbscrews inserted through the slots. From the collector the sewage passes into

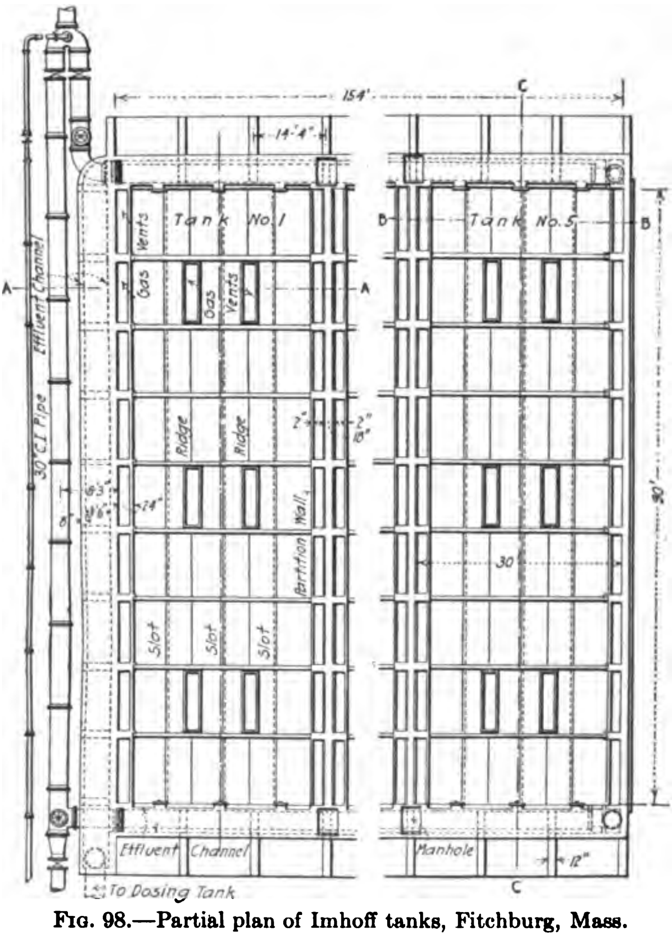


FIG. 98.—Partial plan of Imhoff tanks, Fitchburg, Mass.

an effluent channel leading to the control house of the trickling filters. The design of the tank was prepared to give the sewage 2 hours' detention in the sedimentation chamber with each tank serving 4000 persons.

The solids settling through the sewage in the sedimentation chamber collect in the annular V-shaped space at the bottom and slip through the slot there into the sludge chamber below. The special feature of this

slot is the manner in which it is guarded by a projecting ring on the wall of the sludge chamber, so that bubbles of gas cannot pass through it into the sedimentation chamber. The sludge chamber was designed to retain accumulated solids to the amount of 1 cu. ft. per capita. The depth of the lowest part of the chamber below the surface of the sewage in the sedimentation chamber is 25.5 ft. During the digestion of the sludge in this chamber a large amount of gas is given off, as explained in Chapter VI, which passes off through the central gas vent. The conical upper part of the sludge chamber is sometimes called the scum chamber because large quantities of scum are often formed at certain periods in the operation of Imhoff tanks. The sludge is removed through a pipe by

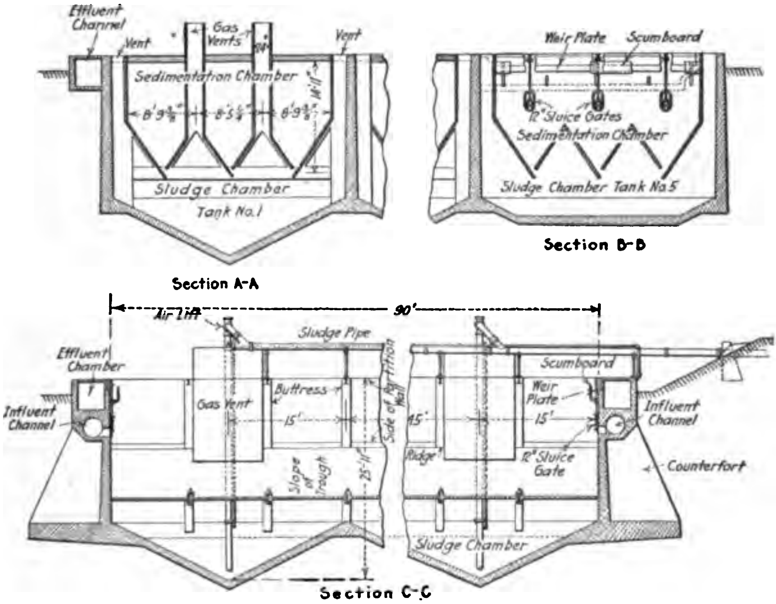


FIG. 99.—Sections of Imhoff tanks, Fitchburg, Mass. (The lines on which the sections are taken are given in Fig. 98.)

gravity when the outlet gate is opened. This outlet is 4.85 ft. below the surface of the sewage in the sedimentation chamber.

An example of the horizontal-flow type of tank is furnished by Figs. 98 and 99, showing an installation at Fitchburg, Mass., built from the plans of David A. Hartwell, City Engineer, and Harrison P. Eddy, Consulting Engineer. This plant, which was one of the first designed in the United States although it was not put into operation until 1914, consists of five 90 × 30-ft. tanks placed side by side. There is an inlet conduit and effluent channel at each end, so that the direction of flow can be reversed through the tanks.

The sewage enters each tank through three 12-in. sluice gates, one for each trough in the sedimentation chamber. In most of the early American Imhoff tanks only 1 or 2 troughs were used over a sludge chamber. The sedimentation chambers were designed to detain the sewage for 3 hours when flowing at the rate of 125 gal. per capita daily from a population of 55,000. The velocity in the tanks under such a condition is about 30 ft. per hour. The velocity of entrance does not exceed 2 ft. per second. At the foot of the tank the settled sewage falls over adjustable metal weir plates into the effluent channel.

The solids slip through the 3 slots into the sludge chamber, which was built large enough to hold the sludge accumulating for 6 months at the rate of 0.007 cu. ft. per capita daily. The gas vents are arranged in 4 lines along each tank and their area is 17 per cent. of the total area of the tank. The sludge must be lifted to beds higher than the tanks, and as raising it with centrifugal pumps would tend to remove the gases which seem to be essential for the characteristic drying of the material, it is handled by air lifts. Attention is called to the fact that the bottom of the sludge chamber consists of three hoppers; an alternative plan usually adopted in Germany is to use independent wells instead of hoppers, with a continuous sedimentation chamber over two or more of them.

Diffusion between Chambers.—The sludge chamber in all Imhoff tanks is intended to retain its contents in a stagnant condition, without any circulation of liquid through the slots opening into it. For this reason, it is considered particularly desirable to design and operate such plants so that no fluctuations of sewage level will occur in the tanks, for it is believed that such fluctuations, particularly if sudden, will cause differences of hydrostatic pressure and, consequently, surges of septic sewage up through the slots. If considerable fresh sewage enters the sludge chamber, there is a strong probability that hydrogen sulphide will be evolved, according to Emscher District experience.

One of the main purposes of the original tanks of this type was to keep the sewage flowing through the sedimentation basins as nearly fresh as practicable, and the mixture of the liquid from the sludge chamber with the contents of the sedimentation chamber would, it was believed, prevent attaining this purpose. It is mainly for this reason that the installations in the Emscher District show such careful attention to grades and other features controlling the elevation of the sewage during both direct and reverse flow through the tanks. Whether this occasional mixture of a little septic liquid will cause so much trouble as the technical staff of the Emscher Commission fears has been questioned in the United States by some engineers, but it is evidently prudent to avoid the chance of trouble from this source if practicable. Another cause of diffusion which can be avoided by careful operation is the

accumulation of sludge too near the slot; it is generally stated that the surface of the sludge should not rise nearer than 18 in. to the slot.

The danger of diffusion is manifestly greatest with 2 or more sedimentation chambers over a single sludge chamber. In such cases care must be taken to have the inlets and outlets designed to give the proper proportion of flow to each sedimentation chamber, and if the operating conditions will be such as to require careful attention from the attendants, it may be wise to follow the system adopted by J. H.. Gregory for such cases, and connect the sedimentation chambers at each end of the tank so as to maintain the same surface elevation in all of them.

The critics of this type of tank have asserted that diffusion, temperature changes, and the gradual increase in the quantity of sludge in the sludge chamber must result in septic liquid passing from it into the sedimentation chambers. While these criticisms are acknowledged to be theoretically correct, the Emscher technical staff holds that they are practically unimportant with good design and competent operation. The area of the slots through which the liquid must pass is very small in proportion to the capacity of the chambers. The specific gravity of the liquid contents of the sludge room is greater than that of the sewage in the sedimentation chambers, and this tends to keep the heavier septic liquid from rising through the slots. This condition is influenced probably by the somewhat higher temperature of the sewage in the sedimentation chambers. According to Dr. Bach, Chemist of the Commission, the most important factor is the sludge collecting over the slot while it is settling to and into it, which forms a sort of automatic seal between the two chambers, hindering by its weight any currents in a direction contrary to that of the settling solids.

It is manifest that some of the liquid in the sludge chamber must be displaced as the solids there increase in volume, but the amount of liquid displaced in proportion to the volume of sludge has aroused some controversy. Dr. Bach states that it is manifest from the results of proper operation of such tanks that the volume of the sludge in the sludge chamber is less than the volume of sludge entering the tank. The parts of the solids which are changed into gas in the sludge chamber manifestly do not displace liquid but escape into the air. As for the rest, Dr. Bach has conducted experiments and studies (*Technisches Gemeindeblatt*, November 5, 1912) which show that in passing through the slots, from the fresh sewage above to the septic liquid below, the volume of the sludge is decreased from 25 to 50 per cent. This was true not only of sewage sludge but, to a somewhat smaller degree, of sediment formed of fresh, washed iron hydroxide and very finely powdered hard coal.

Inlet and Outlet Channels.—The design of the inlet and outlet channels of Imhoff tanks must receive more attention than is usually given to such features of treatment works. Space in the sludge chamber of these tanks is expensive and should be utilized fully, which is accomplished only in horizontal-flow tanks with more than one well by using the inlet and outlet chambers interchangeably, thus reversing the direction of flow through the tanks. This is done at least once a month, and enables the accumulation of the heavier parts of the suspended matter to proceed equally in both wells. Dr. Imhoff usually employs only two wells in series, in order to avoid the unequal distribution of sludge which attends the use of more than this number.[1] The im-

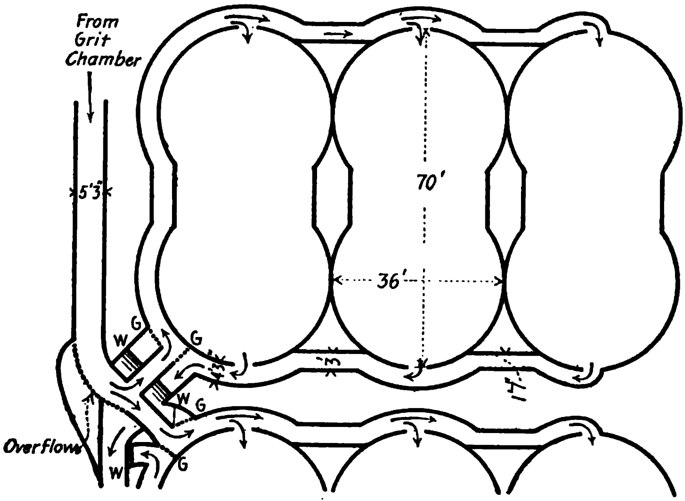


Fig. 100.—Inlet and outlet channels in tanks planned by Imhoff.

portance of the inlet and outlet channels is also increased in the case of Imhoff tanks by the desirability of keeping the surface of the sewage in the sedimentation chamber at the same elevation under all conditions, to prevent septic sewage from rising through the slot, which may occur with fluctuations of surface level, as already explained.

The arrangement preferred by Dr. Imhoff where it is practicable is shown in Fig. 100, prepared from a plan furnished by him to the

[1] An attempt to remedy this condition has been suggested by Charles Hoopes (*Engineering Record*, Oct. 18, 1913), who bases his method on the curve of sedimentation of the sewage during the period of its detention in the sedimentation chamber. This curve he subdivides into the same number of sections as wells, so that during each of these sections the same amount of solids will settle. He then places longitudinal partitions, with gates, in the chambers, so that the velocity of flow through the part of the chamber over each well will give the proper detention period there to insure the desired equal distribution of the solids through the series of tanks.

authors in 1910. The illustration gives the flow lines in one-half of a plant for the sewage of a city of 75,000 persons, where a combined system is in use. Only 2 wells are in series in such a plan, and the use of more than 2 wells in a tank is avoided by working the tanks in parallel. If greater capacity was needed in the unit than the 3 tanks afforded, another tank could be added by making the necessary changes in the inlet and outlet channels. The operation of the tanks is fixed by the elevation of the crest of the weirs, *W*, and the position of the 4 gates, *G*, except when storm water is running off through the overflow. In the latter event there will be some increase in the surface elevation at the inlet to each unit.

The arrangement adopted in 1913 by George W. Fuller in the plant

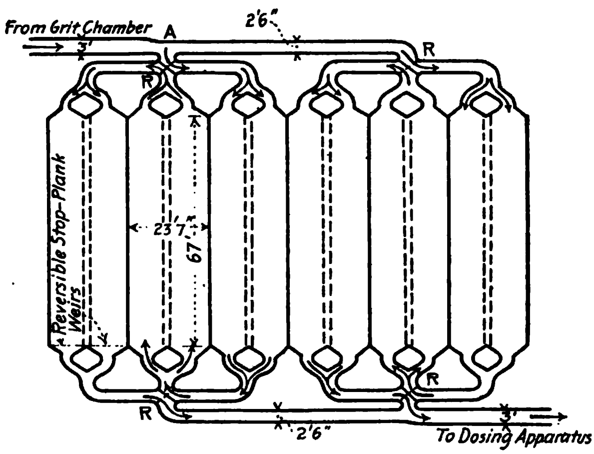


FIG. 101.—Inlet and outlet channels, Schenectady Imhoff tanks.

at Schenectady, N. Y., is entirely unlike that in the design just described. Here there are 6 tanks (Fig. 101), each having 5 wells or compartments with openings in the walls between the sludge chambers, so that the sludge can flow from one to another. The length of travel in each of these tanks is 67 ft., while in Dr. Imhoff's tanks it is 70 ft. Instead of reversing the flow through all the tanks by operating them all in parallel, the Schenectady system groups the tanks in 2 sets of 3 each, with the direction of flow always the same in the center tank of each set but occasionally reversed by stop-planks at *R*, *R*, *R*, *R*, in the outer tanks of each set. It is also practicable to operate all the tanks in parallel. The operation of the tanks is controlled by these stop-planks and by movable plank weirs at the outlet of the last tank which is in series in each set for the time being.

The inlet and outlet arrangements at the plant at Fitchburg, Mass., are shown in Figs. 98 and 99.

In designing the inlet channels of the Imhoff plant at Baltimore, thin longitudinal walls were placed in such position that 1 of the sub-channels does not serve more than 4 tanks. Frank stated in *Engineering Record*, July 4, 1914, that this subdivision was made because it is easier to distribute a given flow properly among a few units than among many, and experience with distribution problems has shown that 4 tanks are not too many to be served by 1 distributing channel. The general arrangement is indicated in Fig. 97 and the following notes from Frank's article:

"In subdividing a given channel, the total difference in water level in its length is increased, but this increase may be neutralized by increasing

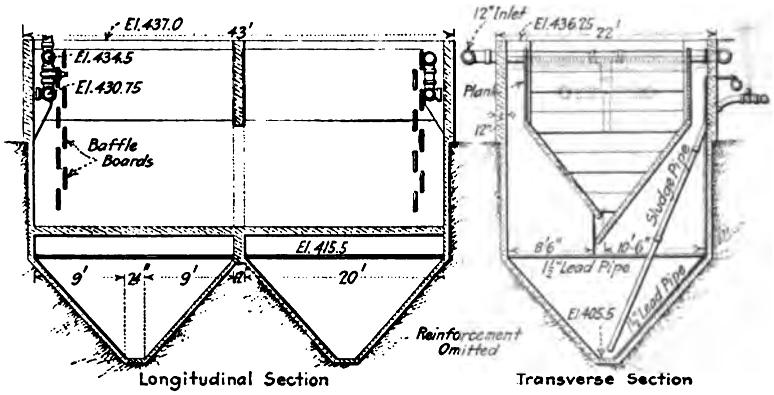


FIG. 102.—Vertical sections of Imhoff tank at Lebanon, Pa.

the total width of channel. This was done in the present instance. The whole distributor system has been so computed hydraulically that the elevation of the water surface at the initial point of subdivision is practically the same in all the sub-channels. This has resulted, of course, in sub-channels of different widths. The distributor channels have, in addition, been provided throughout with adjustable distributor wings so that when the plan is taken into operation the whole system may be regulated to the desired nicety."

The width and depth of the inlet and effluent channels should be such that there is no danger of sedimentation in them. The subject of depositing velocities is discussed in Volume I, pages 108 and 115, and as pointed out there the minimum safe velocity is 2 ft. per second for sewage from separate systems and 2½ ft., or preferably 3 ft., where the sewage contains grit. If the tanks are designed to treat some storm

water as well as sewage,[1] care must be taken to see that none of the outlet weirs of the tanks is submerged by the effluent in the lower end of the effluent channel during periods of heavy discharge. If the free flow of the sewage from the tanks over the weirs is retarded, the height of the water in the tanks will be made to fluctuate, which is considered undesirable by the Emscher District engineers.

Scum boards should be provided in front of all weir outlets, as in the case of septic tanks. The practice in the Emscher District is to submerge them 12 to 16 in. and let them rise 12 in. above the surface. Special attention is directed to placing them so as to avoid any increase in the currents in the lower part of the settling chamber.

The inlet and outlet details in the tank built at Lebanon, Pa., from the plans of James H. Fuertes are shown in Fig. 102. According to *Engineering Record*, May 4, 1912, screened sewage can be admitted at either end through two submerged orifices, which are directed against the adjacent end wall so as to cause the incoming liquid to spread out before beginning its flow to the other end. A set of lattice baffles is provided to assist in stilling the entrance currents. These inlets also serve interchangeably as the suctions of pumps which lift the sewage to trickling filters.

In some of his designs with multiple sedimentation chambers, Hyde has used triangular weirs. In one instance the notch was 4 ft. long and 15 in. deep. As his tanks have special baffles (Chapter XX) in front of both inlets and outlets, to render the flow uniform through the tank, the use of a notch weir is not likely to cause local currents, and it avoids mere trickles of effluent such as take place over broad weirs at very low flows.

Design of Sedimentation Chamber.—The length of the sedimentation chamber, or its radius if a radial-flow tank is used, is determined by the detention period necessary to secure the removal of the settling solids. American engineers have usually chosen from 2 to 3 hours, and the Germans consider 1 to 2 hours enough. After the detention period has been fixed, the length of the tank is chosen, being short for a low velocity and long for a high velocity of flow. The length of the tanks in the Emscher District varies from 25 to 100 ft. and is governed to a considerable extent by the desire to restrict the maximum rate of flow in tanks supplied by combined sewerage systems to 320 to 470 ft. per hour. The heavy discharge usually takes place with a three-fold dilution of the sewage by storm water, and if a greater dilution occurs the storm overflows come into action and the amount of sewage reaching the plant is

[1] In the Emscher District, storm water is admitted to tanks until its quantity is a predetermined multiple of the dry-weather sewage. The excess is then turned into the stream, either directly or after passing through storm-water basins. After a storm, the sludge in such basins is pumped into the channel leading to the tanks.

not increased. The rate when only dry-weather sewage is flowing will be about 108 ft. per hour.

After the length has been selected in the manner outlined, the other dimensions of the sedimentation chamber are obtained by selecting the depth and width which will best suit all the local conditions. This step in the design is not so simple as it seems at first, because German experience has shown it to be desirable to have the horizontal velocity gradually decrease toward the lower part of the sedimentation chamber, in order that there may be no currents to prevent the settling solids from passing through the slot into the sludge chamber. This consideration also affects the depth and position of the baffles used in long Imhoff tanks, for deep baffles will cause a disturbance in the currents in the parts of the chamber where it is most desirable to have steady flow.

As it is very desirable for the settling solids to move steadily to and through the slots, the inclined sides of the bottom of a settling chamber must have a sharp inclination and a very smooth, hard surface. Even with great care in construction, it is necessary in some cases to clean them with rubber squeegees, for soft, sticky material accumulates on them, which will cause undesirable conditions in the chamber unless it is removed. For the same reason, it is desirable to have the top details of the tank such that this cleaning can be thoroughly done, if needed, either from permanently fixed platforms or temporary plank walks.

The slopes for the bottom of the Emscher District sedimentation chambers range from 1.2 on 1 to 1.5 on 1. Where only a single sedimentation chamber is employed, as in the tank[1] at Lebanon, Pa. (Fig. 102), it runs through the center of the tank and the sludge chambers are located on each side of it. In this case the vertical walls of the chamber are formed of 2-in. planks and the bottom of reinforced concrete slabs. This makes a relatively light chamber. In many cases reinforced concrete is used for the vertical sides as well as the inclined bottom.

The concrete slabs used for the sides and bottom of sedimentation chambers are built either on forms like most slabs reinforced with bars, or on metal lath by plastering it with cement mortar. The plaster is applied either by hand or by pneumatic apparatus. Inasmuch as the inclined slabs may collect solids which must be pushed down them, they must have greater strength than is needed to support their own weight and that of a small quantity of submerged solids. They must also be stiff enough to remain free from cracks. Sufficient experience has not yet (1915) been gained to show whether cast or plastered slabs are preferable.

The slot at the bottom of the sedimentation chamber should be

[1] This tank is designed to have a detention period of 2½ hours when treating 500,000 gal. of sewage daily. The sewage is maintained at a constant level in it by the device described in Volume I, page 709.

from 6 to 12 in. wide, depending upon the size of the chamber and the character of the sewage. The horizontal overlap to prevent gases passing through the slot should be at least 8 in.

Theoretical Quantity of Sludge.—The size of the sludge chamber is based primarily upon the amount of solid matter which is expected to settle out of the sewage during the assumed maximum period between removals of sludge, a subject already discussed on pages 50, 51 and 358.

In the Emscher District it is customary to consider that twice as much suspended matter will be deposited from sewage from a combined system as from that from a separate system. This quantity is estimated on a per capita basis instead of in parts per 1,000,000, because Imhoff believes that more accurate estimates of sewage solids are possible when this course is followed. It is not a serious matter in the district in question, because it is customary to estimate sewage treatment requirements for only 5 years in advance. Additional tanks are readily constructed as required, if the original plan provided room for them. If the original estimates of settling solids were too large, the only result will be an operating capacity somewhat more in accord with American ideas of necessary surplus than with those of the Emscher Commission. If the solids were underestimated a new tank will be required in less than 5 years. The average amount of sludge where combined sewers are used is assumed by Imhoff at 0.007 cu. ft. per capita daily, and with separate sewers at 0.0035 cu. ft.

As some chemical wastes act as precipitants, a special investigation is desirable wherever they are turned into the sewage. They also have a decided influence on the best period of detention of the sewage in the tanks. At Essen Nord, for example, the large amount of iron salts in the sewage makes 1 hour the maximum desirable detention period, owing to the coagulation that takes place. Where the sewage is fresh and strictly domestic, a detention period of 2 hours is generally most satisfactory in the Emscher District, and 1.5 hours is probably the average at all of the plants. Imhoff is opposed to a stay of more than 2 hours for German sewages, on account of danger of septic conditions arising in the sedimentation chambers. The earlier Imhoff tanks built in this country were usually designed for detention periods about 50 per cent. longer than the practice in the Emscher District, but a tendency to reduce them became evident later.[1] The periods selected in American designs bring the rate of flow far below the 108 ft. per hour for dry-weather flow, and 324 ft. for flow with three-fold dilution by storm water,

[1] The following are examples of the detention periods in early American Imhoff tanks: Akron, 2 hours; Albany, 3; Atlanta, 3; Baltimore, 2; Chambersburg, 2⅙; Fitchburg, 3; Lebanon, 2½; Madison-Chatham, 2; Philadelphia, Pennypack Creek, 2; Rochester, 1¼; Winchester, Ky., 2.6; Winters, Cal., 3.

adopted by Imhoff. A rate of 27 to 34 ft. per hour results from designs by Dr. Hering; 16 ft., Fuertes; 25 ft., the authors; 16 ft., Fuller. The rate in the large plant at Rochester, designed by Edwin A. Fisher and Emil Kuichling, is 87 ft. per hour, while that in the large plant at Baltimore, designed by Hendrick and Frank, is only 7 ft. per hour. These rates are given by Prof. Peter Gillespie in a paper on "Methods of Treatment of Sewage Sludge" read before the Canadian Society of Civil Engineers on Dec. 17, 1914.

The volume of sludge for which storage space must be provided is usually estimated empirically, but approximate estimates can be made from the amount of suspended matter in sewage. It seems probable from experiments at Chicago by George M. Wisner and Langdon Pearse and at Cologne by Steuernagel that a determination of the extent of sedimentation in a quiescent liquid during a period of a few hours gives results practically the same as those obtained when the liquid passes at low velocity through a horizontal-flow tank. The Cologne experiments were outlined in Chapter X, page 358. The Chicago experiments are described as follows in a "Report on Industrial Wastes from the Stock Yards and Packingtown," made in 1914 by Wisner and Pearse:

"The apparatus used in our experiments consisted of a cylindrical galvanized iron can 2 ft. in diameter and 9 ft. deep, fitted with taps and nipples projecting inside the can to the center, so that the samples drawn were taken from points in a vertical line. Crude sewage was admitted rapidly at the bottom, keeping the contents thoroughly stirred during the filling. The run was assumed to start on the completion of filling. Samples were taken from each of the taps as soon as the can was filled, and a composite sample made to represent the crude sewage. The can was filled to a depth of 8 ft. 6 in.

"Samples were withdrawn for analysis 18 in. below the surface at time intervals of 5, 10, 15, 20, 25, 30, 40, and 50 minutes, 1, 2, 3, 4, 6, and 12 hours. Samples taken at the end of 12 hours, at depths of 7 ft., 7.75 ft. and 8.25 ft., showed a considerably smaller decrease in suspended matter over the results obtained at depths of 18 in., and in some cases an actual increase was recorded.

"To make clearer the relation between sedimentation time and percentage removal of suspended matter, the removals have been plotted (Fig. 103), the results of the 39th Street experiments being added for comparison. To show the effect of varying amounts of suspended matter in the crude sewage, the results have been averaged and plotted by groups according to the initial suspended matter.

"The quiescent experiments indicate that a removal of from 54 to 93 per cent. of the total material in suspension may be attained by 2 hours' quiescent settling, while at the end of 12 hours the removal varied from 55 to 97 per cent., according to the original content of suspended matter. An average removal of 76 per cent. was attained at the end of 2 hours, while the removal at the end of 12 hours averaged 79 per cent., which probably

represents approximately the percentage of settling solids. The removal at the end of 2 hours is appreciably better than in the tanks with the same theoretical detention period, but the former results are raised by the inclusion of a number of tests from sewage much higher in suspended matter than the average throughout the day, and the percentage removals were correspondingly greater. With identically the same sewage, a slightly greater removal might occur under quiescent conditions."

These results, as plotted in Fig. 103, are of particular interest because they show the relative behavior in sedimentation of a weak sewage like

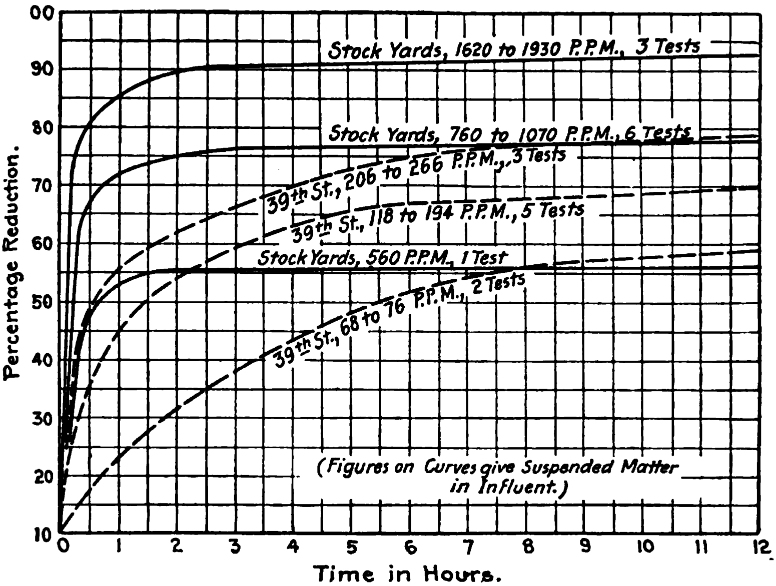


FIG. 103.—Removal of suspended matter by quiescent settling, Chicago.

that at 39th Street, and a very heavy sewage like that of the Stock Yards. The early tendency toward a 3-hour detention period for dilute American sewage is justified by the form of the 39th Street curve.

The results accomplished with an average detention period of 2½ hours at Atlanta, Ga., are given in Table 32, page 167.

The sludge which is collected from sewage differs in the weight per unit volume of the solid matter as well as in the percentage of the suspended matter which has settled in a given period of time. There is a tendency among engineers to compare sludges by expressing their characteristics as those of similar sludges with 90 per cent. moisture, which is readily done by means of Fig. 104, from Webster's report on the Philadelphia experiments. The specific gravities of most sludges of which

analytical records have been examined by the authors lie within 1.02 and 1.07, with 90 per cent. moisture. The exceptions to such a relation were too erratic to make their consideration necessary. Sludge of 1.02 specific gravity at 90 per cent. moisture contains very light suspended matter, such as might be expected from weak domestic sewage. A specific

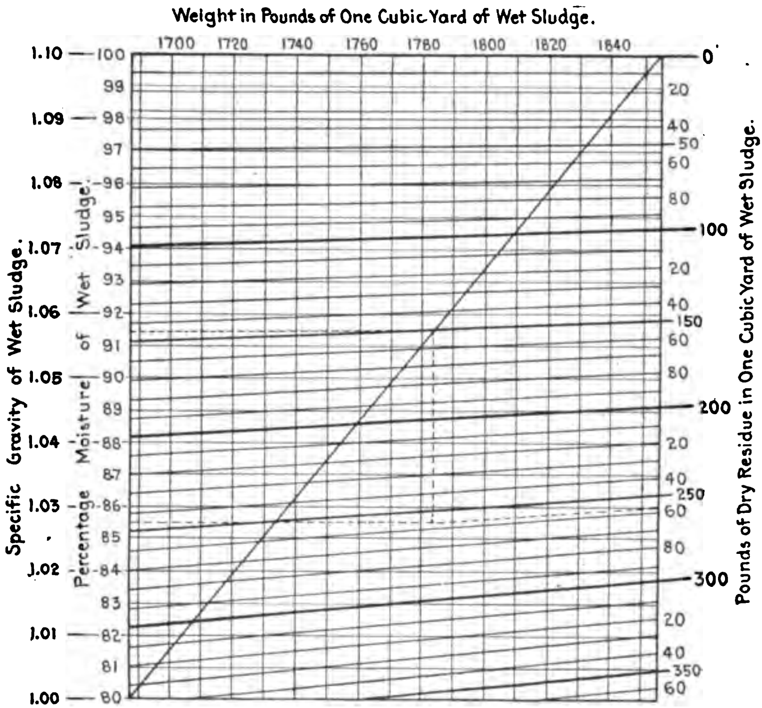


FIG. 104.—Relation between specific gravity, percentage of moisture and weight of dry solids in 1 cubic yard of wet sludge.

Example of Use.—Assume that the sludge is found to contain 85.5 per cent. of water and has a specific gravity of 1.057. From the 1.057 point on the scale of specific gravity, at the left, follow a horizontal line to the diagonal. Directly above the intersection, on the top line, will be found the weight of 1 cu. yd. of such sludge, 1783 lb. Drop a vertical line from the same intersection point and extend to meet it a horizontal from 85.5 on the scale of percentage of moisture. From their intersection, follow the sloping lines to the right to 258 on the right-hand scale, giving the pounds of dry residue in 1 cu. yd. of sludge.

gravity of 1.07 with 90 per cent. moisture is indicative of very heavy solids, such as those entering combined sewers from macadam streets in times of storm. Probably 1.03 will prove a fairly accurate figure to assume for average sewage from strictly separate systems, and 1.05 for average sewage from combined systems.

TABLE 93.—VOLUME OF SLUDGE OF DIFFERENT SPECIFIC GRAVITIES AND PERCENTAGES OF MOISTURE OBTAINED FROM 100 LB. OF SETTLED DRY SOLIDS

Percentage of moisture	Cubic yards of sludge having specific gravity, with 90 per cent. moisture, given in column head					
	1.02	1.03	1.04	1.05	1.06	1.07
99	5.922	5.917	5.912	5.906	5.901	5.896
98	2.955	2.950	2.945	2.939	2.934	2.929
97	1.966	1.961	1.956	1.950	1.945	1.940
96	1.467	1.462	1.457	1.451	1.446	1.441
95	1.175	1.170	1.165	1.159	1.154	1.149
94	0.977	0.972	0.967	0.961	0.956	0.951
93	0.827	0.822	0.817	0.811	0.806	0.801
92	0.729	0.724	0.719	0.713	0.708	0.703
91	0.647	0.642	0.637	0.631	0.626	0.621
90	0.581	0.576	0.571	0.565	0.560	0.555
89	0.527	0.522	0.517	0.511	0.506	0.501
88	0.482	0.477	0.472	0.466	0.461	0.456
87	0.444	0.439	0.434	0.428	0.423	0.518
86	0.403	0.398	0.393	0.387	0.382	0.377
85	0.383	0.378	0.373	0.367	0.362	0.357
84	0.359	0.354	0.349	0.343	0.338	0.333
83	0.337	0.332	0.327	0.321	0.316	0.311
82	0.318	0.313	0.308	0.302	0.297	0.292
81	0.300	0.295	0.290	0.284	0.297	0.274
80	0.284	0.279	0.274	0.268	0.263	0.258

Note.—The sludge in the column headed 1.02 is of the character to be expected from small strictly domestic separate systems, where, through infiltration or high water consumption, the sewage is very weak. The sludge in the column headed 1.07 is from very strong sewage delivered by a combined system, with but little of the mineral matter removed by grit chambers.

The curves shown in Fig. 105 have been used by the authors with satisfactory results in estimating the removal of the suspended matter in sewage by sedimentation.

Water Content of Decomposed Sludge.—While the engineer has some information to help him estimate the amount of settling solids in different classes of American sewage, there is very little to help him estimate the probable percentage of moisture in the sludge. An examination of American records of septic, sedimentation and Imhoff tanks indicates that the probable percentage of moisture is highest in sludge with solids of light weight, ranging from 91 to 98 per cent.; with

28

averge weak sewage (third column of Table 93) the range is about 90 to 96; with average strong sewage (fifth column of Table 93) the range is about 87 to 93.

The Imhoff tanks in the Emscher District yield sludge much more dense than ordinary sludge. Just how much of this density is due to protracted storage and how much to storage in deep tanks must still be determined by careful observation. The subject deserves careful study, for it is probably more important to the designer than most features of these structures; the great difference in the volume of sludge due to a small change in the percentage of moisture in it is clearly shown in Table 93. With this explanation of the danger of relying too heavily on Emscher data until their applicability to American conditions is

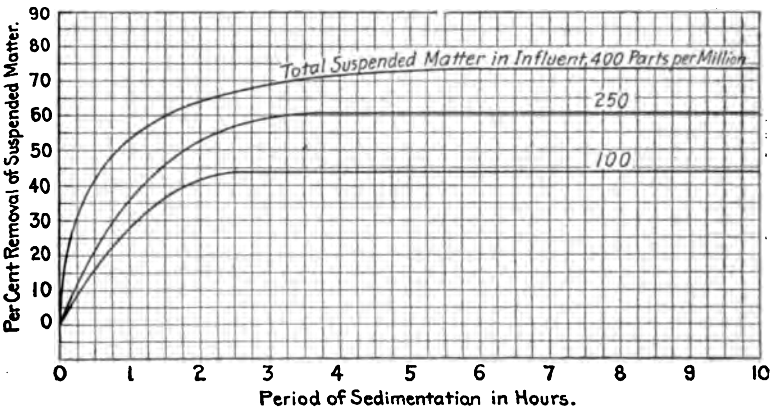


FIG. 105.—Removal of suspended matter in large settling tanks. (These curves are based on the data plotted in Fig. 80, page 370.)

proved, the following information regarding the water content of the sludge of a number of German plants is given here. At Essen Nord, where combined sewage is treated, the fresh sludge was found to contain 94.6 per cent. of moisture, and the decomposed sludge 84.4 per cent. At Rechlinghausen the decomposed sludge from combined sewage contained 83.4 per cent. of moisture. In both cases large amounts of ironworks wastes were discharged into the sewers, acting as precipitants, so that the sludge contained voluminous hydrates. In the case of plants receiving only domestic sewage, the percentages of moisture in the decomposed sludge were: Aplerbeck, 77.2; Holzwickede, 72.2; Schwerin, 75.3; Sodingen, 77.1; Westhausen, 70.3.

The meager American results do not show such low water content. At the Stock Yards testing station at Chicago, experiments were made in a tank 9 ft. in diameter and about 17 ft. deep from the surface of the sewage

to the bottom of the sludge chamber. This tank was first operated as a radial flow tank, with a settling capacity of 2240 gal. and a sludge capacity of 4160 gal., giving a ratio of 1 : 1.85. The gas vent occupied 8.8 per cent. of the total tank area. The tank was remodeled for horizontal flow, with a settling capacity of 1190 gal. and a sludge capacity of 4390 gal., giving a ratio of 1 : 3.69. The gas vent area was increased to 34 per cent. of the total tank area. The composition of the sewage is given in Table 29. The average proportion of moisture in the sludge after the tank was operating well was 91.5 per cent. and the specific gravity was 1.02. The period required for ripening the sludge was apparently about 7 months. The digestion of the total solids when the tank was operating satisfactorily was apparently about 35 per cent. (Report on Industrial Wastes from the Stock Yards, page 76.)

Experiments were conducted at Philadelphia under the direction of Webster with Imhoff tanks 5 and 10 ft. in diameter. The composition of the sewage is given in Table 45. The sludge had an average of 82.6 per cent. of moisture, and the minimum content was 75 per cent. It was drawn only from the most dense part of the accumulation, and the percentage of moisture in the whole of the sludge was believed to be much higher than 82.6. (Report on Work at Sewage Experiment Station, page 163.)

At Worcester, Mass., experiments with an Imhoff tank 12 ft. square and 22 ft. deep were conducted by Almon L. Fales for about 18 months under the direction of Matthew Gault, Superintendent of the Sewer Department. The character of the sewage is given in Table 46, which does not, however, give the high iron content due to wastes from wire mills; 19 per cent. of the dissolved iron and 25 per cent. of the suspended iron were removed in the experiments, which low reduction was attributed to a colloidal condition. The percentage of suspended matter removed ranged from 52 with a detention period of 2.3 hours to 68.7 with a period of 4.1 hours. The percentage of moisture in the sludge ranged from 89.3 to 92.54. The lowest specific gravity, 1.006, was for the sludge with 10.7 per cent. of solids, which was attributed to the relatively large amount of gas in well-digested sludge. The average specific gravity of the sludge was 1.016 and the average percentage of moisture was 91.86. The average digestion of total solids was 45.5 per cent. (Report on Experimental Treatment of Sewage, 1912, page 32.)

At Akron, Ohio, experiments were conducted in an Imhoff tank 8 ft. in diameter and 16 ft. deep by Harry B. Hommon. The sedimentation capacity was 2038 gal. and the sludge capacity 2610 gal., giving a ratio of 1 : 1.28. The average detention period was 2.3 hours. This tank was operated for 8½ months and removed from 26.8 to 61.9 per cent. of the suspended matter, the average removal being 48.4 per cent. The sludge in the tank contained 74 per cent. of moisture and had a specific

gravity of 1.12. About 25 per cent. of the total solids removed were estimated to have been liquefied. (Report on Sewage Treatment, 1912, page 30.)

At Atlanta, Ga., C. C. Hommon imformed the authors, 30 samples of sludge from the Proctor Creek plant showed an average specific gravity of 1.02, 87.05 per cent. of moisture, 39.1 per cent. of volatile matter, 60.9 per cent. of fixed matter, 1.25 per cent. of nitrogen and 6.11 per cent. of fat. Twenty-five samples of sludge from the Peachtree Creek plant showed an average specific gravity of 1.02, 90.2 per cent. of moisture, 36.6 per cent. of volatile matter and 63.6 per cent. of fixed matter.

The depth of tanks in the Emscher District averages about 30 ft., because Imhoff has been convinced that better drying sludge is obtained at this depth than in shallower tanks. Tanks with only 20 ft. depth have been built under his direction, however, where the difficulties of construction make deeper tanks very expensive, but in such cases, he has stated repeatedly, he does not expect to obtain such dense sludge as the standard tanks furnish.

Frank and Fries stated in *Engineering Record*, October 25, 1913, that if shallow tanks are well proportioned they should give as good sludge as deep tanks. They base this belief on the assumption that the rapidity and thoroughness of decomposition of the sludge depends in part on the intensity of the stirring of the sludge by the ebullition of gas. Each cubic foot of decomposing sludge they regard as giving a certain intensity of ebullition, and the deeper the tank the greater would be the agitation of the sludge and hence the rate of its decomposition. To obtain the same grade of sludge with a slower rate of decomposition in a shallow tank, the latter must have a larger capacity than a deep tank in order to afford a longer detention period. This increase in size makes it important, according to this theory, to design the details of the shallow tanks so as to obtain a good distribution of the sludge over the bottom of the digestion chamber.

Experiments were conducted by H. W. Clark and Stephen DeM. Gage at the Lawrence Experiment Station in tanks 17 and 30 ft. deep, filled with sludge from a settling tank at the station. The sludge in these tanks was found to contain 350 and 360 parts per 1,000,000 of hydrogen sulphide, and to have the same amount of moisture, 91.3 per cent. The total reduction of suspended solids was about 19 per cent. in the shallow tank and about 30 per cent. in the deep one. Fermentation was more active in the deeper tank. The reduction of fats was 49 and 79.3 per cent. in the shallow and deep tanks, respectively, the reduction in total organic nitrogen was 28.9 and 21.7 per cent. and the reduction in insoluble organic nitrogen was 30.3 and 22.7 per cent. The most important feature of the tests, as recorded, concerns the effect of shallow and deep storage in preparing sludge for draining; sludge from the

shallow tank contained 76.6 per cent. of water after draining for 10 days while that from the deep tank contained only 65.9 per cent. after similar drying. (Report Mass. St. Bd. Health, 1913, page 274.)

Experience in California has convinced Prof. Charles Gilman Hyde that Imhoff tanks need not be more than 16 to 20 ft. deep, provided proper provision is made for a more voluminous sludge than from deep tanks. In this connection it is desirable to repeat that Imhoff does not lay so much stress on the inoffensive nature of the sludge from deep tanks as on the rapidity with which it is dried. This rapid drying he considers due to the large amount of entrained gas held in the sludge by the weight of the sewage above it. The greater the depth the heavier this pressure and the larger the volume of entrained gas, according to Imhoff. As rapid drying means economical disposal of sludge, he lays great stress on deep tanks as conducive to economical operation. Inoffensive operation at greater expense is practicable with tanks only 20 ft. deep, according to the German experience. American experience has not yet (1915) been sufficiently extensive and thorough to furnish light on the subject.

Sludge Storage Capacity Required.—From what has been stated, an estimate of the storage space required in the sludge chamber can be made by the following process. From analyses of the sewage, or by study of the sources of the sewage when analytical records are lacking, the amount of suspended matter is first ascertained. As an example, it may be assumed at 175 parts per 1,000,000, which is not far from the average for domestic sewage collected by small separate sewerage systems. This amount is equivalent to 1460 lb. of suspended matter per 1,000,000 gal. of sewage. The part of this which will settle in the tank will vary with the character of the sewage. A combined sewage rich in the wastes of steel works is given a 1-hour detention period in Germany and a weak domestic separate sewage is given 2 hours. In the example, it will be assumed that the detention period is 3 hours, during which period 60 per cent. of the suspended matter passes from the sedimentation chamber into the sludge chamber. This amounts to 876 lb. per 1,000,000 gal. of sewage. The estimate of the amount of this suspended matter which is digested in the sludge chamber should be conservative, in view of the very little definite information available on the subject. In the example, it will be taken as 25 per cent., giving 657 lb. of suspended matter as likely to be found in the digested sludge for each 1,000,000 gal. of sewage passing through the tank. The percentage of moisture in the sludge must be assumed very cautiously, for it has a very marked effect on the estimated volume of the sludge. In the example, 85 per cent. will be chosen, which is shown by Table 93 to be equivalent to 0.378 cu. yd. of sludge per 100 lb. of suspended matter; this means that $6.57 \times 0.378 = 2.48$ cu. yd. of sludge storage

capacity must be provided for every 1,000,000 gal. of sewage passing through the tank during the sludge digestion and storage period, which may be taken as not over 7 months in ordinary temperate climates without very hot dry seasons of long duration. This assumption results in about 528 cu. yd. space required for 7 months' storage for a daily flow of 1,000,000 gal. of such sewage.

This estimate can be checked by means of the Emscher District experience, which calls for an average sludge storage capacity of about 1 cu. ft. per capita for combined sewage and 0.75 cu. ft. for domestic sewage. The designs are based on about 0.007 cu. ft. of sludge per capita per day with combined sewers and 0.0035 cu. ft. with separate sewers, so it is apparently customary to allow the sludge from strictly domestic sewage to remain in the tank somewhat longer than that from combined sewage. Gregory prepared Table 94 for the Metropolitan Sewerage Commission of New York to facilitate the preparation of estimates on this basis. Inasmuch as the German sludge has an average water content of 75 per cent. and in the example the water content has been assumed as 85 per cent., the estimated volume of 528 cu. yd. must be reduced to its equivalent in the more compact German sludge. This is done by using the formula

$$\frac{V_2}{V_1} = \frac{100 - P_1}{100 - P_2}$$

where V_2 is the volume desired, V_1 is the volume under the actual conditions, P_1 is the percentage of solids in the original sludge, and P_2 is the percentage of solids in the modified sludge. This formula gives the volume of the sludge with 75 per cent. moisture as 317 cu. yd. By referring to the gagings of domestic sewage in Volume I, pages 180 and 188 to 193, it will be seen that a flow of 1,000,000 gal. a day may be taken as the quantity to be expected from a population of about 10,000 persons. The Emscher practice would call for sludge storage of about 278 cu. yd. for this population. In view of the different conditions in American and German cities, it is customary to consider that the German allowance for sludge space should be increased to give a basis for American designs. Frank and Fries have recommended an increase of 50 per cent. (*Engineering Record*, November 8, 1913.)

Table 95 gives the amounts of solids removed from a number of Emscher District plants in 1913, as communicated to the authors by Imhoff. From information furnished by the superintendent of the sewage treatment works at Batavia, N. Y., the authors estimate that the average daily flow of sewage in 1914 was 1,670,000 gal. The first sludge was drawn from the Imhoff tank on June 17 and the last on Dec. 1, the total amount being about 691 cu. yd., or 1.13 cu. yd. per 1,000,000 gal. The sludge per day was 51 cu. ft., and as the population of the village was

TABLE 94.—VOLUME OF SLUDGE FROM SEWAGE, COMPUTED BY THE IMHOFF METHOD

(John H. Gregory)

Sewage flow, gallons per capita daily	Volume of sludge per 1,000,000 gal.			
	Separate system 0.0035 cu. ft. per capita daily		Combined system 0.007 cu. ft. per capita daily	
	Cu. ft.	Cu. yd.	Cu. ft.	Cu. yd.
50	70	2.6	140	5.2
60	58	2.2	117	4.3
70	50	1.9	100	3.7
75	47	1.7	93	3.5
80	44	1.6	88	3.2
90	39	1.4	78	2.9
100	35	1.3	70	2.6
110	32	1.2	64	2.4
120	29	1.1	58	2.2
125	28	1.0	56	2.1
130	27	1.0	54	2.0
140	25	0.93	50	1.9
150	23	0.86	47	1.7
160	22	0.81	44	1.6
170	21	0.76	41	1.5
175	20	0.74	40	1.5
180	19	0.72	39	1.4
190	18	0.68	37	1.4
200	17	0.65	35	1.3

about 12,500, the amount of sludge produced was about 0.004 cu. ft. per capita daily.

Another method of estimating the volume of sludge space has been developed by Kenneth Allen ("Sewage Sludge," page 228), who has proposed the following formulas:

$$\text{Storage (combined sewage) in cubic feet} = 10.5\ PD$$
$$\text{Storage (domestic sewage) in cubic feet} = 5.25\ PD$$

where P is the population in thousands and D is the detention period in days. The formulas are based on sludge with 80 per cent. moisture. In the assumed case, with a population of 10,000 and a detention period

of 213 days, this formula calls for a sludge space of 414 cu. yd. If the amount of moisture is assumed to be 85 instead of 80 per cent., the required space will be 552 cu. yd., or a trifle more than that estimated by the method suggested by the authors.

TABLE 95.—SOLIDS REMOVED FROM SEWAGE IN EMSCHER DISTRICT PLANTS IN 1913

Place	Population	System	Sewage, gal. per capita daily	Liquid rotted sludge		Dried sludge, cu. yd. per. mil. gal. sewage	Scum, cu. yd. per mil. gal. sewage	Grit, cu. yd. per mil. gal. sewage	Screenings, cu. yd. per mil. gal. sewage
				Gal. per capita daily	Cu. yd. per mil. gal. sewage				
Essen-Nord....	200,000	Combined	56	0.058	5.1	2.23	0.56	0.18	0.06
Bochum.......	165,000	Combined	80	0.035	2.2	1.49	0.05	0.19	0.06
Essen-Nordwest	100,000	Combined	63	0.080	6.2	3.72	0.40	0.47	0.03
Gelsenkirchen..	74,000	....	46	0.098	10.4	5.89	1.00	0.25	0.07
Herne-Nord....	55,000	....	69	0.042	3.0	1.69	0.14	0.57	0.16
Recklinghausen	30,000	Combined	79	0.029	1.8	1.43	0.05	0.08	0.06
Oberhausen....	25,000	....	74	0.045	3.0	1.62	0.02	0.11	0.06
Stoppenberg...	22,600	....	68	0.070	5.2	3.20	......	0.70	0.14
Sodingen......	8,900	....	62	0.073	5.8	4.14	0.07	0.77	0.04
Lutgendortmund........	5,000	....	28	0.046	7.9	5.75	0.50	0.58	0.14
Teutoburgia....	4,000	Separate	43	0.038	4.1	2.91	0.23	......	0.23
Westhausen....	3,500	Separate	68	0.033	2.5	1.68	0.13	......	0.08
Gr. Schwerin...	3,050	Separate	23	0.052	11.6	0.73	0.44	......	0.29

In providing this amount of storage for the sludge care must be taken that the shape of the tank will permit all the room allotted to the sludge to be actually available for it, and that the sludge does not approach within 18 in. of the slot. With some sewage the ebullition in the sludge chamber and the formation of scum is very active at times, particularly in protracted hot weather. Hyde has observed Imhoff tanks under such conditions when there was no sludge in the chamber and all the solids were in the thick scum. An unusual case was observed during Fales' investigations at Worcester, when sludge entered the sedimentation chamber, something to be avoided in practical operation. The report of this experience is as follows:

"The percentage of solids appears to be lowest after warm seasons, which are conducive to bacterial action. Such a condition was particularly noticeable upon the examination of the sludge on September 20, at a time when the level of the sludge unexpectedly rose to the level of the slot. At this time the top sludge appeared to be very thin for a depth of 6 ft. The accumulation of scum to such an extent that the gases of decomposition did not have a free vent doubtless aggravated this condition. An unusually

voluminous evolution of gas for several days following the removal of the scum on December 16, 1912, showed the retarding action of the scum on the escape of the gas. The release of the gas was doubtless instrumental in an increase in solids from 7.9 per cent. on December 17 to 10.7 per cent. on January 18 following. Recently, when the sludge had risen to such a level that sludge scum was forming on the sedimentation chamber, the removal of scum allowed the excess of gas to escape, lowering the level of the sludge so that no further contamination of the sedimentation chamber resulted for several weeks." (Report on Experimental Treatment of Sewage, 1912, page 28.)

Gases from Imhoff Tanks.—The gas from one of the Atlanta tanks was found to have the composition given in Table 96.

TABLE 96.—RESULTS (IN PERCENTAGES) OF ANALYSES OF GAS FROM IMHOFF TANK VENTS AT PEACHTREE CREEK PLANT, ATLANTA, GA.

(Analyses by C. C. Hommon; *Engineering News*, April 2, 1914)

	Sample 1	Sample 2	Sample 3	Sample 4	Average
Carbon dioxide........	4.7	5.2	4.4	4.2	4.6
Oxygen...............	0.3	0.5	0.6	0.4	0.4
Hydrogen sulphide.....	0.0	0.0	0.0	0.0	0.0
Methane	85.3	82.8	84.2	....	84.1
Hydrogen.............	9.7	8.2	7.9	....	8.6
Nitrogen.............	0.0	3.3	2.9	....	2.1
	100.0	100.0	100.0	4.6	99.8

Investigations by Fales at Worcester furnished the information given in Table 97 concerning the gases from an Imhoff tank, a sedimentation basin, a septic tank and a digestion tank into which the sludge from sedimentation basins was discharged daily.

TABLE 97.—GASES FROM DIFFERENT TYPES OF TANKS AT WORCESTER, MASS.

(Report on Experimental Treatment of Sewage, 1913, page 23)

Tank	Sedimentation, per cent.	Septic, per cent.	Sludge digestion, per cent.	Imhoff, per cent.
Carbon dioxide......	49.6	5.9	44.0	27.1
Heavy hydrocarbons.	....	0.5	0.6	0.2
Oxygen............	0.0	0.8	0.0	0.3
Carbon monoxide....	0.0	0.0	....	0.2
Methane...........	40.1	75.2	35.7	43.8
Hydrogen..........	....	0.3	0.0	10.2
Nitrogen, by difference..............	10.3	17.4	19.7	18.1

The absence of hydrogen sulphide from the gases given off from Imhoff tanks, except when fresh sewage enters the sludge chamber for any reason, has caused much speculation. The late Dr. Spillner, chemist of the Emscher Board, was of the opinion that hydrogen sulphide was due mainly to the septicization of dissolved organic and colloidal matter in the sewage, and this action was not permitted in properly operated Imhoff tanks. Fuller has stated his belief that hydrogen sulphide was formed in the sludge chamber, but the active ebullition in the contents of this chamber dispersed the gas through a large volume of liquid and prevented its escape into the atmosphere.

Shape of Sludge Chamber.—There has been a tendency in the United States to construct the sludge chamber as a hopper-bottomed tank. In the Emscher District, on the other hand, cylindrical tanks with conical bottoms are generally used. This preference is based on the lower cost there of cylindrical over rectangular tanks. An example of this construction is shown in Fig. 106, one of the units of the Essen-Nordwest plant. The 6 wells of this plant are intended for 30,000 persons, or a maximum of 6,604,000 gal. in 24 hours of combined sewage, according to information furnished by Imhoff. As the dry-weather sewage of this district is given as 63 gal. per day in Table 95, the amount of storm water for which provision is made is apparently about 2.5 times the volume of sewage.

Sludge Removal.—The sludge chamber should be designed so that the oldest portion of the sludge can be drawn off first, which calls for inclined bottoms sloping to sumps. In the deep tanks used in the Emscher District, where the construction sometimes became relatively very expensive as the depth increased beyond certain limits, special attention was devoted to developing a form of bottom on which the sludge could be handled satisfactorily without incurring unnecessary expense for excavation. It was found that a slope of 1 on 2 was practicable if some provision was made for flushing the sludge with water if it failed to slip. The water is admitted through a ring of 1½ to 2-in. perforated lead pipe at or near the top of the slopes, and occasionally through another ring around the sump at the bottom, or a single jet. The perforations in the rings used in the Emscher District are about 5⁄32 in. in diameter and 20 in. apart.

The sludge is usually removed through an 8-in. pipe. In Germany this is always carried up straight until the top is above the surface of the sewage, so that in case it becomes clogged with sludge, which occasionally forms a mass of the consistency of thick molasses within it, the material can be stirred by a jet of water through a cock tapped into the cap on the end of the pipe. American engineers have not been disposed to regard the straight pipe as necessary, although its desirability is acknowledged. Where a straight pipe is used the sludge is not discharged from the end

but through a curved branch with a valve, which is so located, if possible, that the outlet of the branch is from 4 to 6 ft. below the level of the sewage. This head of sewage is enough to force the sludge, after it is loosened by jets of water from the perforated pipes, up through the sludge pipe and its branch to a drain to sludge-drying beds. Where a gravity discharge is impracticable, the sludge may be raised vertically by an air lift to a sufficient elevation to flow by gravity to the drying beds, Fig. 99. The air lift is used in order to avoid the suction of an ordinary pump, which, it is feared, would remove too much of the gas which is a very important part of Imhoff tank sludge, causing it to dry rapidly and thoroughly.

A gravity pipe line for Imhoff tank sludge should have a slope of at least 15 per cent. in order to avoid danger of stoppage. (Frank and Fries, *Engineering Record*, November 8, 1913.)

After a tank in the Emscher District has gone through the initial ripening process, a portion of the sludge within it is drawn off about every 4 weeks, except when winter weather makes it undesirable to place sludge on the drying beds. American tanks have usually been designed so that no sludge need be removed from them during the winter. In the operation of the tanks it is necessary to keep the sludge at least 18 in. below the slots, as before stated, and its upper elevation can be measured by lowering through a gas vent a weighted flat board or disk of sheet steel by means of stout cord until it rests on the sludge. Experience shows that all the sludge should never be withdrawn, for if this is done it is necessary to repeat the tedious and sometimes troublesome initial ripening process.

Scum Chambers.—The space in the decomposing chamber above the slot is often called the scum chamber, because the ebullition of the gases toward the end of the ripening period, which usually is about 4 weeks long in the Emscher District and a shorter time when the climate is warmer, lifts considerable quantities of sludge into it. An experienced attendant can often determine the character of the action of a given tank by watching carefully the formation of scum in the scum chambers. The capacity of the scum chambers in the Emscher District averages about half that of the sludge chambers.

The gas vents, the parts of the scum chambers extending to the top of the tank, should be large enough not only to enable all the gas to escape but also to permit workmen to enter the sludge chamber when the tank is emptied. The German practice is to make their area from one-fourth to one-third of the area of the sludge chamber, although in some of the tanks in the Emscher District no difficulty has been experienced in working with ratios as low as 1 : 10. American practice was originally toward small ratios, but difficulties with scum have led to the use of larger gas vents. Where the vents are small, the scum forms rapidly, and if the

attendants do not break it up as soon as it collects in appreciable quantities, it may rise over the edges of the scum chamber. Imhoff has recorded an instance of scum rising 6 ft. above the level of the sewage, because the gases could not escape.

Solids which collect on the surface of the sedimentation chambers are skimmed off by attendants and thrown into the scum chambers.

The scum areas should be easy of access to enable the attendants to break up the scum in order to liberate the gases which hold up the solids forming it. After the first few months of service so much gas is sometimes formed quickly that most of the sludge is lifted into the scum chambers. This condition should be avoided if possible, and it can often be prevented, when it seems about to occur, by stirring the sludge with jets of water from the perforated pipes at the bottom. Imhoff considers that such stirring is also helpful in assisting the decomposition of the organic matter by removing the toxins caused by previous energetic bacterial action. The scum should be removed to the sludge drying beds, if it becomes too voluminous even after stirring the sludge with water jets. Such scum can be skimmed off the liquid with spades, but this is unnecessary in plants operating in a normal manner.

Where the bacterial action is very marked and the ripening period is particularly short, as in warm climates, Hyde is of the opinion that the tanks must be designed so that the scum can not only be broken up by the attendants but also so that it can be pushed down into the sludge chambers. Under such operating conditions he has found that the scum became offensive in odor and not properly digested. If the odors did not disappear when the scum was broken frequently, it was necessary to shovel it out of the gas vents and bury it.

If the sludge remains yellow or gray in color and has the odor of sour milk, it is often due to acid conditions in the tank which can be helped by adding occasionally a little milk of lime to the sewage in the inlet channel.

In drawing off sludge, the rate of withdrawal should be slow in order that the whole mass of sludge may settle and no conical depression exist in the center. In the latter case a considerable amount of relatively fresh sludge will be withdrawn and some of the older well-rotted material left, whereas all the sludge which is still undergoing active decomposition should remain in the chamber to keep up, without interruption, the changes taking place there.

Reconstruction of Septic Tanks.—The possibility of converting sedimentation and septic tanks into Imhoff tanks has been discussed rather unfavorably by some engineers on account of the shallow depth in such reconstructed basins as compared with the Emscher District precedents. An instance of successful reconstruction is afforded by two covered septic tanks at Orange, Cal., which caused much annoyance by their

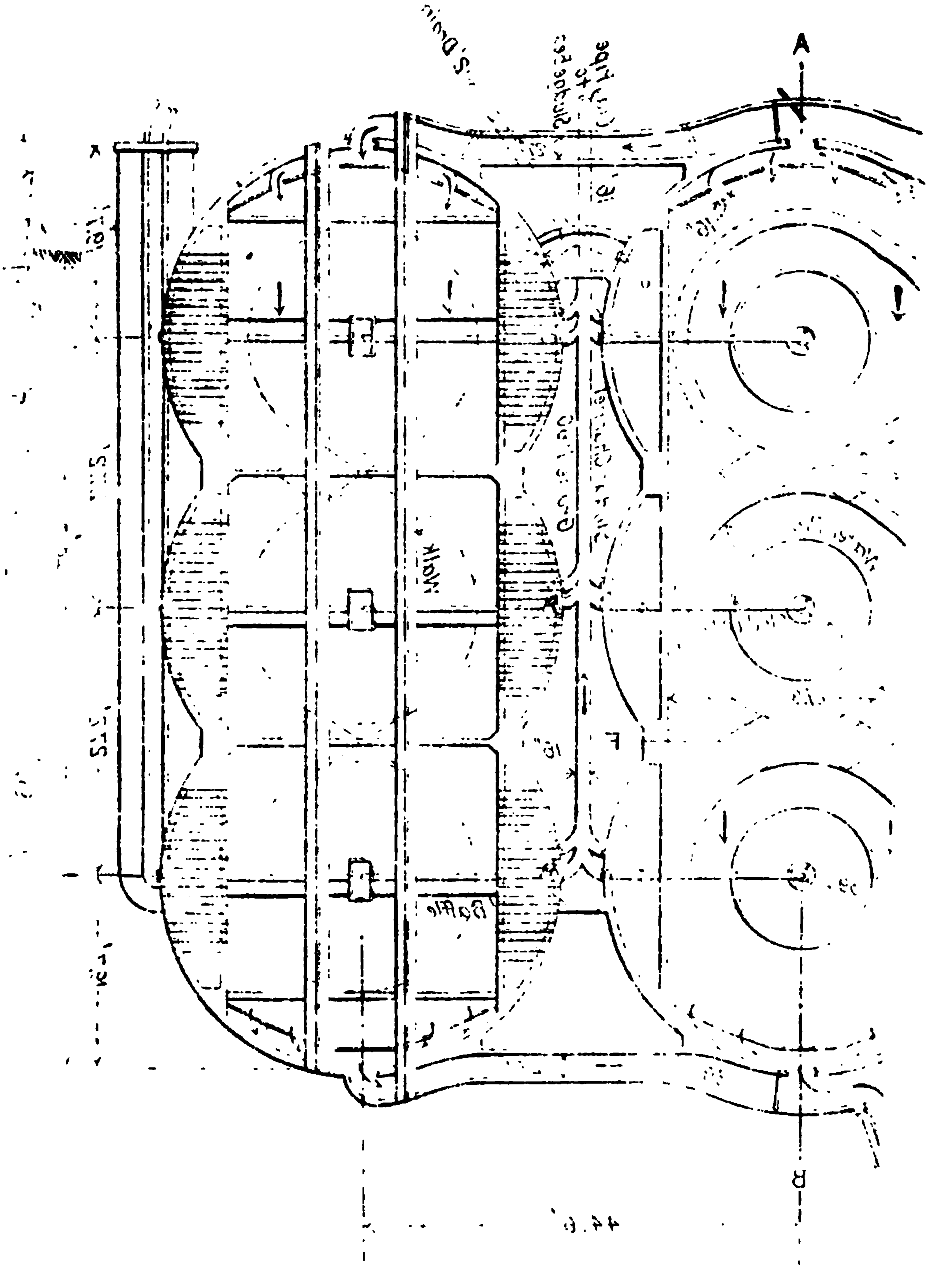
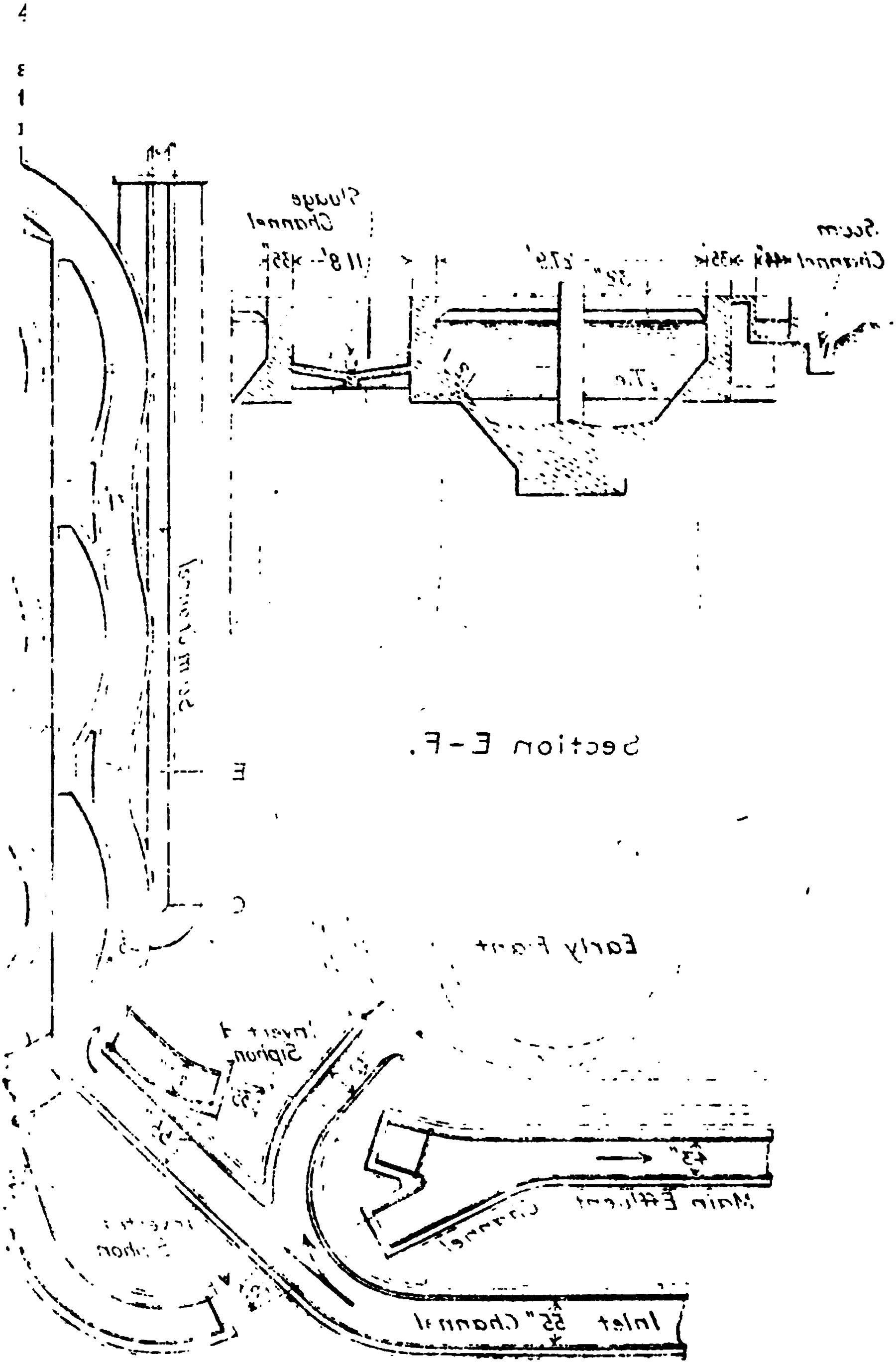


FIG. 100.—Imhoff tanks at Essen-Nordwest.


Sludge Channel
Scum Channel
Section E-F.
Inverted Siphon
Main Effluent Channel
Inlet 55" Channel

odor. As a temporary expedient, Hyde advised their conversion into Imhoff tanks as shown in Fig. 107.

Each septic tank is about 67.5 ft. long, and was subdivided by redwood planks and timbers so as to have 2 sedimentation chambers and 5 sludge chambers. The latter were formed by constructing transverse partitions at 14-ft. intervals, and in the bottom of each bay a multiple header of 4-in. pipe was laid on the concrete floor, with an outlet through the side wall, to drain off the sludge to a sunken pump pit. A baffle was placed in each sedimentation chamber in front of the inlet, but no

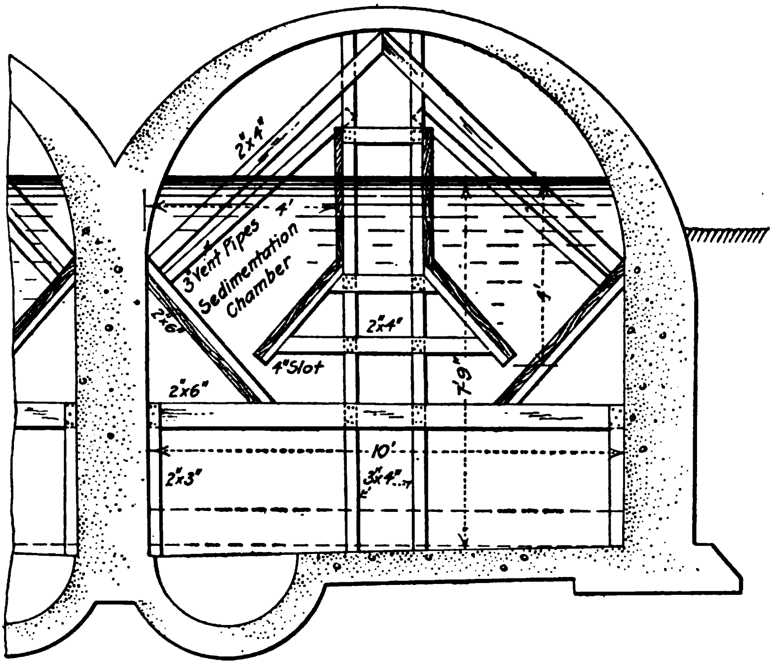


FIG. 107.—Conversion of septic into Imhoff tank, Orange, Cal.

other was used. A manhole about 4 ft. long and 3 ft. wide was cut in the roof over the center of each sludge chamber. The sludge drains by gravity to a centrifugal pump. It is typical dark-brown Imhoff tank sludge and dries readily on the sludge beds to which it is pumped. By keeping the sewage fresh instead of allowing it to become septic, as before, comparatively little odor is detected at the tanks.

THE LETHBRIDGE TANK

A form of two-story tank used in a number of Canadian cities is shown in Fig. 108. It is known as the Lethbridge tank, from the Alberta

city where it was first used, and was introduced there by T. Aird Murray, Consulting Engineer, of Toronto. This plant was described in a paper before the Canadian Society of Civil Engineers on March 4, 1914, by A. C. D. Blanchard, City Engineer of Lethbridge when the tank was

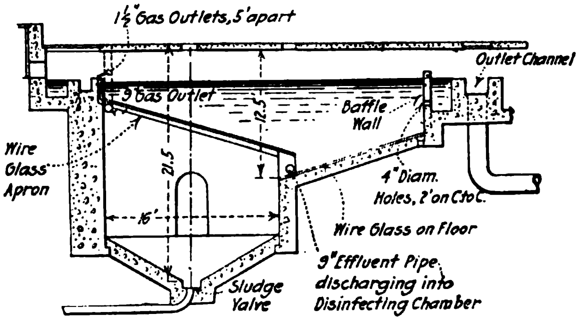


FIG. 108.—The Lethbridge tank.

built. The following information was supplied to the authors by W. A. Adams of Lethbridge.

The rate of flow of the sewage was found by gaging to range from 400,000 gal. per day at night to a maximum of 2,000,000 gal. The normal day rate was 1,000,000 gal. The sewage is weak and fairly

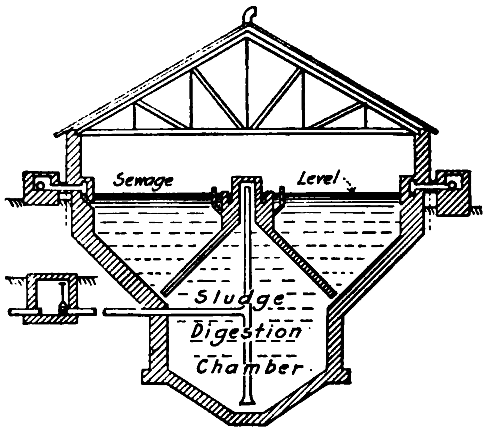


FIG. 109.—Tank for separate sludge digestion, Whitby, Ont.

fresh when it reaches the works, which are designed to treat 2,400,000 gal. daily. It passes through a grit chamber and a rack with 0.5-in. openings before reaching the tank, which it enters over a weir along the entire length of one side so as to have a very low velocity as soon as it

is in the tank. It is drawn off through submerged outlets 4 in. in diameter and 24 in. apart.

The feature of the design of most interest is the method of constructing the sludge chamber and the slot giving access to it. This is shown in the illustration. There are none of the provisions for handling scum which are found in Imhoff tanks. The structure is satisfactory except that the 8½-lb. beams supporting the wire glass were on 5-ft. centers, which gave too wide a span for some of the sheets, which gave way. The sedimentation chamber has a capacity of about 116,000 gal., and the sludge chamber about 75,000 gal., estimated as the volume of sludge accumulating in 3½ months. The detention period is 2½ hours when the tank is worked at full capacity, under which conditions the rate of flow is 130 ft. per hour. The sludge produced is very black and inodorous. Owing to the breakage of the glass in the tanks at Lethbridge, North Toronto, Oakville and other Canadian cities, the slopes of the sedimentation chamber were made steeper and concrete used in place of glass. An example of the new type of construction is shown in Fig. 109, illustrating a tank built in 1914 at Whitby, Ont., from designs by Murray.

CHAPTER XII

CHEMICAL PRECIPITATION

It has been pointed out in Chapter X that sedimentation for 6 to 12 hours will remove from one-third to one-half of the suspended matter in sewage, depending upon the amount and character of that matter. Much of the remaining suspended and colloidal matters can be removed by adding chemical precipitants to the sewage. These form a bulky gelatinous coagulant that gathers up and carries these matters to the bottom of the tank. This treatment, called chemical precipitation, can produce under favorable conditions, a clear, colorless effluent containing little suspended matter. Although considerable dissolved organic matter is said to be removed by this process, the fact that much of the colloidal matter of sewage will pass through filter paper shows that probably the so-called dissolved organic matter removed may have often been colloidal matter. The effluent may be very satisfactory in appearance but is ordinarily putrescible and not comparable with good filtration effluents. The high cost of this treatment has often led to curtailing the quantity of chemicals used, so that results obtained in practice are rarely as good as the method can produce. The greatest field of usefulness for chemical precipitation is apparently in connection with the treatment of industrial wastes.

Chemical precipitation is practised in numerous British cities and was recommended by the Royal Commission on Sewage Disposal as the best preliminary treatment of strong sewage, particularly if it contains brewery or tanning wastes (see page 26). In the United States other methods of preliminary treatment have been generally adopted, but the method is practised on a large scale at Worcester, Mass., and has been employed for a number of years at Providence, R. I. In Table 98 are given the percentages of removal of suspended matter by chemical precipitation at a number of works and experiment stations.

CHEMICALS USED

A great number of chemicals have been tried as precipitants, but of late years only a few have had extensive use—lime, alum, copperas, ferric sulphate and alumino-ferric—and only these will be considered here.

Lime.—The quicklime of commerce varies greatly in quality, and unless the amount of impurities in the material used are known the actual

TABLE 98.—REMOVAL OF SUSPENDED MATTER BY CHEMICAL PRECIPITATION

British cities	Precipitant	Percentage of suspended matter removed	Suspended matter in effluent, parts per 1,000,000	American cities	Precipitant	Percentage of suspended matter removed	Suspended matter in effluent, parts per 1,000,000
Chorley....	Alumino-ferric	91	29	Alliance, O.	Lime	56	20
Dorking...	Lime	82	70	Canton, O.	Lime	44[1]	51
Guildford	Alumino-ferric	75	146	Chicago stock yards	Lime and copperas	77	105
Hendon....	Alumino-ferric	80	55				
Heywood..	Alumino-ferric	82	50	Columbian Exposition	Lime and copperas	68	57
Horfield...	Alumino-ferric	93	25	Columbian Exposition	Lime and alum	70	53
Kingston...	A.B.C.	95	15	Columbian Exposition	Lime, copperas and alum	82	38
Normanton	Alumino-ferric	67	180	Columbian Exposition	Lime, copperas and ferric sulphate	64	50
Rochdale..	Alumino-ferric and sulphuric acid	79	78	Columbian Exposition	Lime and ferric sulphate	78	37
York......	Lime and alumino-ferric	64	77	Columbus, O.	Lime and copperas	52[1]	70
				Columbus, O.	Alum	74	39
				Glenville, O.	Lime	26	50
				Providence, R. I.	Lime	83[2]	47
				Worcester, Mass.	Lime	82[3]	60

[1] As good results were obtained by plain sedimentation. [2] Removal of suspended matter determined by albuminoid ammonia. [3] This figure is an average of 10 years' removal determined by albuminoid ammonia, which is the method of estimate employed at this plant; the percentage removal of suspended matter, as determined by evaporation, was recorded in the annual reports for 1893 and 1894 as 90.67 and 94.45 respectively. In 1903 to 1912 the greatest removal of suspended matter, as measured by the albuminoid ammonia, was 93.7 per cent. and the lowest was 74.7 per cent. It should be added that all of the sewage during this time was not treated chemically, but the strongest parts, requiring the greatest amount of treatment, were submitted to plain sedimentation and intermittent filtration.

amount of active precipitant available is a matter of guesswork.[1] The active part of the lime is the calcium oxide, CaO, but not all of the lime is in this form.

In an investigation of the precipitation process at Alliance, Canton

[1] "During the winter a large number of analyses of lime were made, resulting in the selection of a grade better adapted to sewage treatment, and which, taking into account the quality, has proved to be about 20 per cent. cheaper than that formerly used." (Rept. of Supt. of Sewers, Worcester, Mass., 1892.)

and Glenville in 1906–7, the Ohio State Board of Health found that at the first place hydrated lime with 42 per cent. available calcium oxide was used, at Canton the percentage was 12 to 35, and at Glenville about 62. In an investigation at Columbus, Ohio, Copeland and Sperry found that the percentage of CaO in carload lots of lime from 8 different sources ranged from 77.4 to 94.0. It is possible to keep a more careful control over the treatment of sewage or water by using uniform, rich lime than supplies with 20 to 30 per cent. of impurities.

Testing Lime.—The methods of determining the total and available calcium oxide employed by the Sewer Department of Worcester, Mass., for many years are as follows:

Total Calcium Oxide.—Dissolve 0.2 gram of the sample in dilute (1 to 10) hydrochloric acid, keeping the liquid warm until the lime is dissolved. There will be some insoluble matter, which should be filtered out after neutralizing the solution with ammonia. The filtrate is boiled and ammonium oxalate added until the calcium is all precipitated. The liquid is then kept in cold water until clarified, when it is filtered. The residue is dissolved in sulphuric acid and titrated with standard potassium permanganate solution.

Available Calcium Oxide.—Boil 0.1 gram with a moderate amount of water at least half an hour to render the calcium carbonate insoluble. Titrate with $\frac{1}{10}$ normal hydrochloric acid, using phenolphthalein in an alcoholic solution as an indicator.

The test for available calcium oxide is particularly important, because the presence of any carbonate indicates either imperfect burning or deterioration by air-slaking.

Commercial Lime.—The best lime for sewage treatment is obtained by burning compact, non-magnesian, high-calcium limestone. It is claimed that the most even product is obtained by using gas as fuel, because the temperature of the kiln is under better control. Lime burned with wood is ranked above that burned with coal because wood has a longer flame and lower temperature, which reduces the danger of overburning. Wood-burned lime is usually whiter than the coal-burned product.

Overburning causes a partial fusion of the lumps containing siliceous impurities, which protects part of the calcium oxide from slaking and retards the slaking generally. Such lime is said to be "dead burnt," and when made into mortar the slaking continues and produces blisters, or "popping," on the surface. Underburned lime is of poor quality because of the calcium carbonate remaining in it. All unburned lumps are called "cores," and should be removed if mechanical slaking is adopted.

Lime is sold in bulk in small lots by the bushel of 80 lb. and in barrels weighing about 220 lb. When large quantities are bought in bulk, it is weighed in carload lots on track scales, like coal, and its value ascertained by a determination of the available calcium oxide. At St. Louis,

where the lime for the water purification plant is delivered in cars, a shovelful of lime is taken at the center and each corner of the car about 1 ft. below the surface, and the price is based on the CaO of these samples, in the same way that coal is bought on the basis of its thermal units. In 1910 the City of Columbus adopted the following basis of payment for lime:

"For any carload lot containing 88 per cent. of available CaO the city will pay to the contractor the price per ton stated in the proposal. It is hereby agreed that the city will pay a bonus of 14 cts. per ton for each 1 per cent. by which the available CaO in any carload lot delivered shall exceed 88 per cent., and shall deduct a penalty of 14 cts. per ton for each 1 per cent. by which the available CaO in any carload lot shall be less than 88 per cent."

Small quantities of lime are usually weighed on platform scales, but where overhead conveying systems are employed in handling the lime, a suspended scale may prove more suitable.

Storing Lime.—When quicklime or calcium oxide is left exposed to the air it takes up moisture and carbon dioxide rapidly. This air slaking should be prevented as much as possible in transporting and storing the lime. Provision must also be made for the expansion of the lime should air slaking occur, and on this account the bins must be strong and space left between their walls and the sides of the storehouse. Lime may be stored in bins of matched plank for several months with very little loss of available calcium oxide, except in the upper part, which slakes to a fine powder that seals the lower material. The air in the storehouse should be as dry as it is practicable to keep it. It is particularly important to prevent lime becoming wet, for if this happens enough heat may be generated to set fire to the wood of the bins. On account of the progressive loss of available CaO during storage, the amount present when the lime is actually used, instead of when it is received, may need determination where very careful treatment is required.

At the St. Louis water treatment plant the lime is stored in cylindrical concrete bins 20 ft. in diameter and 47 ft. deep to the apex of the hopper bottom. Each has a thin flat concrete roof and also, below this flat roof, a steel conical roof with its apex at the side of the bin at the point where the lime enters, provided to reduce the air space above the lime to the minimum amount. There is a manhole at the top of the bin and one in the side about 12 ft. below the top. A series of barring holes 6 in. square, closed with doors, runs down each bin, and through them the attendants loosen the lime if it arches or becomes caked. Several 2-in. pipes closed with plugs were built into the conical bottom for the same purpose. Each bin has a capacity of about 10,800 cu. ft., ample to store about 310 tons of lime.

Quantity of Lime Required.—The quantity of lime required for successful treatment varies with the character of the sewage and the method of precipitation which is employed. Normal domestic sewage is slightly alkaline. Where lime alone is the precipitant for the treatment of domestic sewage, it is necessary to add a sufficient quantity to combine with all the free carbonic acid and the carbonic acid of the bicarbonates, producing normal calcium carbonate, which acts as the coagulant. Much more lime is generally required when it is used alone than when sulphate of iron or alumina are also employed.

Where, through the introduction of industrial wastes, mineral acids or acid salts are present in the sewage, these must be neutralized before precipitation can take place and may increase the quantity of lime required.

The quantity of lime to be used in any process of treatment is determined by testing samples of the sewage taken frequently at a point where it is certain that the chemicals added have become well mixed with the sewage. If the test shows that a greater or smaller quantity of chemicals is required, the rate of their introduction is modified accordingly.

The required degree of alkalinity, where lime alone is used for treatment, may be determined by titrating one portion of the sewage with erythrosine and another portion with phenolphthalein. The erythrosine alkalinity must exceed twice that shown by the phenolphthalein test.

If the process of treatment depends upon the addition of copperas, from which hydrate of iron is to be formed, as the effective agent of precipitation, the reaction will be $FeSO_4 + CaO_2H_2 = CaSO_4 + FeO_2H_2$. It is necessary to add sufficient lime to provide a normal carbonate alkalinity. This may be determined by the use of phenolphthalein as an indicator. When an alcoholic solution of phenolphthalein is used for this test a few drops added to a glass of treated sewage will produce a pink color in the sample when enough lime has been added. Only enough lime should be added to the sewage to produce the pink color.

If sulphate of alumina or ferric sulphate is used, the natural alkalinity of the sewage may be sufficient for precipitation, bicarbonate alkalinity being capable of precipitating these salts. Bicarbonate alkalinity is not shown by phenolphthalein, but is indicated by erythrosine, which is used in the presence of chloroform and imparts a pink color to alkaline liquids, but is colorless with neutral or acid solutions.

Methyl orange is a useful indicator for testing some industrial wastes for acid. Half-bound carbonic acid, or bicarbonates, cause it to react acid. Methyl orange indicates an acid condition by a pink color and an alkaline or neutral condition by its natural yellow color. The color imparted is faint and difficult to detect unless the test is carefully conducted, as by placing a little of the indicator in a white dish and allow-

ing drops of the sample of wastes to fall into it from the end of a glass rod. If the yellow persists and no pink spots appear where the drops fall, the wastes are alkaline. A limitation of this indicator is that it may indicate alkalinity even in the presence of sulphates of iron and aluminum which are actually acid salts requiring alkali for precipitation.

If too much lime is used in the treatment of sewage, some of the suspended organic matter will be dissolved and the effluent may be worse than the raw sewage, as was the case with the chemical precipitation plant at Alliance, Ohio, according to the 1908 report of the Ohio State Board of Health. On the other hand, if not enough lime is used, the effluent will not be well clarified, and if from an acid sewage it may remain acid and be poorly prepared for treatment in contact beds or trickling filters.

While an excess of lime causes the solution of some of the suspended organic matter, this result may be considered less objectionable than those following the changes taking place in stale or septic strong sewage during the detention period in a tank, which changes can sometimes be checked in a measure by adding an excess of lime.

Copperas; Ferrous Sulphate.—A large quantity of copperas or protosulphate of iron is obtained in the United States from the liquids used in pickling steel wire and sheets. Large amounts were once obtained from pyrites, and it has been produced by dissolving scrap iron in the sludge acid of petroleum refining. Copperas is used extensively as a mordant, as a disinfectant, in the manufacture of ink and pigments, and in making fuming sulphuric acid. It is obtained as light green crystals having the formula $FeSO_4 . 7H_2O$ which soon becomes covered with a very light brown dust, basic ferric sulphate, when exposed to the air. When large amounts of copper and nickel sulphates are present as impurities, the color is very dark and the salt is termed "black-vitriol."

Ordinary sewage not influenced strongly by alkaline wastes is not sufficiently alkaline to precipitate ferrous sulphate, and lime must be mixed with the sewage, preferably before the iron salt is added.

When added to sewage, ferrous sulphate is changed to ferrous hydrate, which is soluble in water to the extent of about 7 parts per 1,000,000. It is desirable to have the iron finally reach the form of normal ferric hydrate, but experience shows that enough oxygen is seldom present to produce this change. In his experiments at Louisville, Fuller had the following experience with ferrous hydrate in Ohio River water:

"When ferrous sulphate is added to this water, white ferrous hydrate, mostly insoluble, is formed. Very quickly this precipitate passes into solution, due to the action of carbonic acid and resulting probably in the formation of a soluble basic carbonate. When the iron is in this form, the atmospheric oxygen, although present in excess, oxidizes it very slowly and with great difficulty. Furthermore, the iron, when it does reach the oxidized

state, does not form the normal gelatinous ferric hydrate, but a partially granular compound which is some lower hydration of ferric oxide, as nearly as could be learned." ("Water Purification at Louisville," page 381.)

Ferric Sulphate.—Ferric sulphate (persulphate of iron) has been obtained by weathering pyrites in the presence of sulphuric acid and by treating a solution of ferrous sulphate with sulphuric and nitric acid. Its principal use is in sewage treatment. It does not dissolve readily. Average sewage, not strongly influenced by industrial wastes, is sufficiently alkaline to cause precipitation with ferric sulphate. The ferric hydrate which is formed when this sulphate is used as a precipitant, is insoluble and very gelatinous.

Ferrozone is a mixture of crude ferrous sulphate with smaller amounts of ferric sulphate, oxide of iron and sulphates of aluminum, magnesium and calcium, prepared by a patented process from an unusual grade of iron ore found in southern Wales.

Alumino-ferric.—This is prepared from bauxite; at several British sewage treatment works it is manufactured as needed instead of being bought. Crude bauxite is treated with sulphuric acid, resulting in a hard cake containing iron and silica, as impurities, mixed with crude aluminum sulphate.

Sulphate of Alumina.—This is known in the trade as basic sulphate of alumina and is usually spoken of as "alum" by engineers. Its value depends upon the amount of "available" or soluble sulphate in it. This ranges from about 16 to 18.5 per cent.

When added to water, the sulphate is changed by the carbonates and bicarbonates of calcium and magnesium dissolved in the water into little flocculent masses of aluminum hydrate. The carbon dioxide set free during the decomposition of the alum is dissolved in the water while the lime and magnesia become soluble neutral sulphates of calcium and magnesium.

Average sewage, without an excessive amount of acid wastes, is sufficiently alkaline to produce precipitation with sulphate of alumina.

PREPARATION OF CHEMICALS

While it is impracticable to give as much attention to the details of operating small as large plants, the success of the former will depend in no small degree upon the manner in which those in charge are able to utilize on a small scale the information obtained by the more detailed studies of the chemists of the large plants.

As a general proposition, where the coagulant is prepared by hand, the batches are made up for at least 1 day's demands. Alkaline solutions are stored in concrete or steel tanks while acid solutions are stored in wooden tanks, which are preferably lined with lead although con-

siderable service can be obtained from tanks of sound, clear lumber without a lining. Pipes carrying milk of lime may become choked with precipitated carbonate of calcium, and it is desirable to have facilities for flushing them with very weak acid and water. Acid solutions cause deterioration of iron and steel pipes, and lead or lead-lined pipe is preferred to other kinds by some engineers on that account. It is sometimes helpful to blow steam or air through the feed pipes. The danger of pipes choking is much reduced by clarifying the solutions, except milk of lime, by sedimentation or by straining off the suspended matter.

Alumino-ferric.—Probably one reason for the popularity of alumino-ferric as a precipitant in England is the ease with which it is used in comparison with any method involving the preparation of milk of lime or chemical solutions for dosing small volumes of sewage. All that is necessary is to place a block of the alumino-ferric in the sewage channel with its top projecting above the level of the sewage. The greater the flow of sewage the larger the amount of precipitant dissolved. It is occasionally used in solutions, however, as at Horfield and Hendon, but in a very simple way. A box with a perforated bottom is charged each morning with enough of the alumino-ferric for the day's needs. Water is allowed to drip on the chemical, dissolving it slowly, and the solution is piped off to the sewage channel. The solution is strongest during the first few hours after the fresh charge of chemical is placed in the box, and the strength of the sewage is also greatest about this time, so that the solution and the sewage fluctuate in quality in about the same way. At York, Wellesden, Ealing and Friern Barnet alumino-ferric is used in combination with milk of lime, which gives a better effluent than the former alone, as was found to be the case at Dorking during some experiments conducted by the Royal Commission on Sewage Disposal. The amount of alumino-ferric used with average sewage in several English cities was given by the Commission as 70 to 130 parts per 1,000,000 and the dose for strong sewage as 75 to 250 parts per 1,000,000.

Slaking Lime.—Lime does not dissolve readily in water and on this account it is best applied to sewage in the form of milk of lime. This is prepared by slaking quicklime in a small amount of water and allowing it to stand over night, or longer if possible. This is then mixed with about twenty times its weight of water. Sometimes a thinner mixture is used.

Lime of a fair quality will take up about one-third of its weight of water in slaking, but poor grades will not require so much. The amount necessary for slaking varies and should be determined by experiment. A quick-slaking lime will require more water than hard, close-grain grades. In general, the lime will increase from 50 to 100 per cent. in bulk in slaking.

The temperature during slaking is an important index of the character of the operation. It should be as near 200°F., or just below the boiling point, as possible. If the temperature stays below this point, too much cold water has been added and the slaking is less vigorous and thorough. If not enough water is added the temperature will become too high and there will be danger of "burning" the lime."

Worcester Practice.—At Worcester, Mass., the only American city where chemical precipitation has been continued uninterruptedly and successfully since it was begun, the sewage is passed through a grit chamber and then receives its dose of milk of lime, as will be explained later. It then passes into 6 roughing tanks, measuring about 100 × 66.7 × 7 ft. deep, (5 ft. deep below the weir, on which the sewage is about 18 in. deep), and afterward into 10 finishing basins, 166.7 ft. long, 40 ft. wide and 7 ft. deep, (Fig. 18). Two of these tanks are used for plain sedimentation as a preliminary treatment for a part of the sewage which is filtered. The sludge is drained by gravity to a sludge well, from which it is pumped by a Shone ejector into two 20 × 66.7 × 11-ft. covered storage basins. Advantage is taken of every opportunity to allow sludge to settle quietly in the basins, and the supernatant liquid which is drawn off after such periods of subsidence is filtered on beds of gravel used solely for that purpose and for filtering the filter-press liquor. The sludge is screened upon leaving these storage basins, from which it is forced by stuff pumps into the filter presses. Lime is added to the sludge before it reaches the ejector. The press liquors are filtered through gravel beds and the sludge cake is dumped.

The experience with lime at Worcester dates from 1890. At the outset, the lime, after being crushed and screened to take out the coarse material, was mixed with sewage in elliptical vats, 5 × 8 × 7 ft. deep, each containing 2 vertical shafts with paddles for stirring. In 1893, 2 iron tanks were introduced, each 16 × 8 × 3.5 ft., with perforated iron pipes at the lowest points in 3 shallow troughs forming the bottom. The lime was dumped into the tank, water added, and then compressed air was turned into the pipes. It escaped through the perforations and caused a violent agitation of the liquid. A great deal of trouble was experienced with the clogging of the pipes and finally the bottoms of the tanks were cemented over and the mixing was done by hand. Lump lime was dumped into the tanks and just enough water added to cause proper slaking when the contents were stirred by hand.

The amount of milk of lime added to the sewage at this plant is determined by frequent tests with phenolphthalein, and consequently there is no necessity of keeping the dose of chemical at a uniform strength. The milk of lime which is discharged from a tank immediately after the first slaking is stronger than that used later because water is added to the tank as its contents are drawn down. The dilution of the lime re-

quires a larger amount of the liquid to be added to the sewage, but there is no difficulty of ascertaining when this is necessary from the indications of the phenolphthalein test.

The quantity of lime used at a place like Worcester, Mass., where the dose of precipitant is changed constantly in accordance with the needs indicated by phenolphthalein tests, varies widely from day to day and year to year. In 1893, 1233 lb. per 1,000,000 gal. were used as an annual average, but as experience was gained in managing the plant and intermittent filters were put into service for treating the strongest sewage by plain sedimentation and filtration, the amount of lime required was

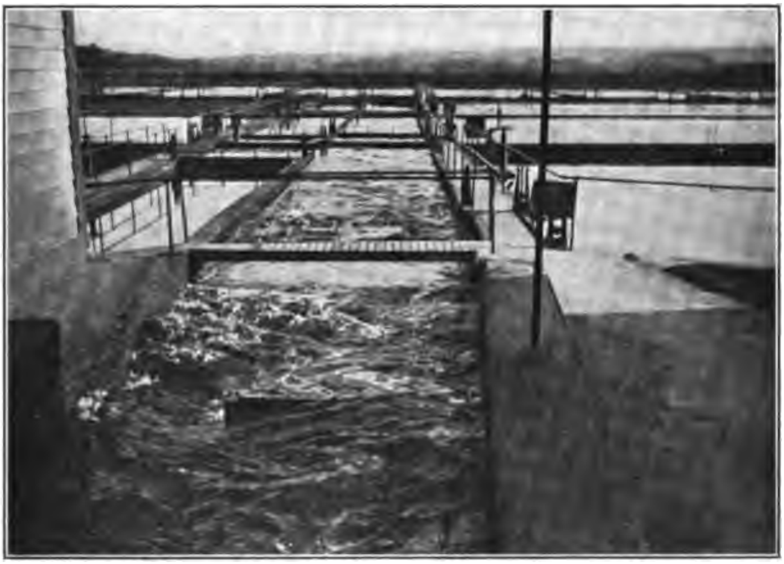


Fig. 110.—Baffled mixing channel or "fish ladder" leading to precipitation basins, Worcester, Mass.

gradually reduced, and in 1912 but 905 lb. per 1,000,000 gal. were used. A rather interesting fact was observed during 1908, when there was a widespread depression in business. This resulted in a great reduction of output in the local foundries and wire works and a corresponding reduction in the quantities of waste acid discharged into the sewer, so that only 871 lb. of lime per 1,000,000 gal. were required.

The mixing of the precipitant and sewage takes place in a channel provided with baffles projecting alternately from each side, which causes violent agitation of the sewage (Fig. 110). Such a baffled channel is called a fish ladder, as it is in common use to permit fish to surmount dams when they run up rivers to spawn.

St. Louis Practice.—The best results are obtained when lime is first reduced to powder before it is mixed with water or tank effluent to form milk of lime. Considerable loss may result from imperfect grinding or breaking up of the lumps. At the water-treatment plant of the St. Louis water works, according to Jacobs (*Jour. Assoc. Eng. Socs.*, March, 1910), the lime from the cars is discharged by chutes into a gyratory crusher fixed to deliver a ½-in. product. A considerable part of the lime is much finer than this. It is carried by conveyors to the storage bins described on page 451, and is drawn off from the bottom as needed, to conveyors which take it to a steel storage tank in the top of the building, large enough to hold about 30 tons.

Directly below the storage tank are 3 slaking tanks of ⅜-in. steel plate. They are 7.5 ft. in diameter, 3.5 ft. deep, with covers of ⅛-in. steel. The stirring apparatus within each of them is driven through a vertical shaft rising from the floor below. The lime is weighed in automatic hopper scales on its way from the storage tank to the slaking tank. The scales weigh from 40 to 100 lb. by ¼-lb. increments, and may be set to dump automatically at intervals of 1, 2 or 4 minutes, which allows the delivery of lime to be adjusted to any point between 10 and 100 lb. per minute. The temperature of the contents of the tank is kept close to 200°F. while a steady stream of fresh water is fed into the tank and an equal amount of milk of lime is drawn off through an overflow pipe. The main features of this practice were explained in *Engineering News*, January 11, 1912, by E. E. Wall, as follows:

"After long and exhaustive experiments, this (the thorough and continuous hydration of lime) was found to be attained by keeping the temperature of the fresh water supplied from 90° to 100°F., and adding from three and one-half to four times as much water as lime by weight. The temperature of the fresh water-supply is kept up by passing it through the coils of a heater tank into which the milk of lime at 200° is drawn. There are two of these tanks located on the floor below the mixing tanks. The milk of lime overflows from the heater tanks into a diluting and cooling box, where sufficient cold water is added to bring its temperature down to about 100°, whence it is pumped by a centrifugal pump into the main well, where it is mixed with the water discharged from the main pumping station."

Storing Copperas.—At St. Louis, copperas is stored at the water purification plant in reinforced concrete bins similar to those used for lime and described on page 451. Considerable trouble was once experienced there by the caking of the copperas, which permitted only the material forming a loose cylinder in the center of the tank to drop out of storage. A device was rigged up by which a U-shaped loop of heavy chain is lowered into the central cavity from a movable horizontal beam above the tank, and the beam is then rotated about a vertical shaft, dragging the chain around the surface of the caked sulphate.

Dissolving Ferrous Sulphate.—Sugar sulphate[1] of iron can be dissolved rapidly if water is forced upward through the chemical, overflowing at the top. One way of utilizing this fact is to employ a wooden tank with a false bottom perforated with ⅛-in. holes. The sugar sulphate is placed on this bottom, and water under pressure is admitted below the bottom, passing upward in jets through the perforations and stirring up the grains of the precipitants.

In the large plant for treating the St. Louis water supply (*Engineering News*, January 11, 1912), the copperas is discharged from a storage bin, into which it is delivered daily from the receiving bins by conveyors, into 2 dissolving tanks of ¼-in. steel plate, each 5 ft. in diameter and 5.25 ft. deep. The copperas is crushed to the size of granulated sugar and then fed by an automatic device into the dissolving tanks. The fresh water enters the bottom of the tanks and the solution overflows through a pipe at the top, no agitating apparatus being needed inside the tank to accelerate the dissolving process.

Lime and Acid Salts.—Where lime and an acid salt are used as precipitants, the lime is generally added first. It reacts with the free and half-bound carbonic acid present, neutralizes any acid salts or free acid, and leaves a slight caustic alkalinity to react further with the acid precipitant added after the milk of lime has been well mixed with the sewage.

Providence Practice.—The plant at Providence, R. I., went into service in 1901, and was designed after a careful study of the operating methods and results at Worcester. There are 4 roughing tanks, two about 100 × 101 ft., one 105 × 112 ft. and one about 105 × 123 ft., with an average depth of 11.87 ft. below the outlet weirs; and 16 finishing tanks, each 60 × 115 ft., with an average depth of 8.67 ft.

The inlets to the roughing tanks are 10 ft. wide and closed by cypress gates raised by worm gears. The channels from these basins are arranged to permit the effluent from any of them to be delivered to any finishing tank. The inlets to the finishing tanks are 16 in. in diameter, with sluice gates, and the sewage on passing through them is deflected to each side in an inlet trough with a rounded lip over which the sewage passes in a thin sheet. This detail was adopted to aerate the sewage as well as to produce a uniform flow through the tanks. This lip has the same elevation as the outlet weir of the tank. The tanks can be emptied through floating outlets. The sludge from all the tanks passes to a sludge well 30 ft. in diameter and about 11 ft. deeper than the bottoms of the roughing tanks. The sludge is screened, and pumped by Shone ejectors into sludge reservoirs, with floating weirs for drawing off the supernatant liquid. The sludge is forced into filter presses by compressed air.

[1] Sugar sulphate of iron received the name "sugar" because it is produced in granulated form rather than large crystals, its usual form.

While chemical precipitation was practised at Providence,[1] from 3000 to 4000 lb. of lime were mixed at a time. According to Julius W. Bugbee, chemist and superintendent of the works, the mixing was done in reinforced concrete tanks 16 ft. long, 8 ft. wide and 3 ft. deep, with the bottom 4 in. lower at one end than the other. A 2 × 4-in. bronze sluice gate was set in the end wall at the low point and controlled the discharge through an 8-in. vitrified pipe. After water had been added to the lime, the slaking mass was stirred by heavy mortar hoes until all lumps had disappeared and the lime was of uniform texture. It was then allowed to stand until required. When the contents of a tank were being discharged they were stirred with hoes or a small water jet. The amount of the dose was determined by testing the sewage frequently below the point of injection of the lime.

The results and cost of chemical treatment at Providence are summarized in Table 99, together with information regarding later operation of the plant by plain sedimentation with subsequent disinfection of the effluent. During 1905 to 1910, inclusive, the supplies for precipitation

TABLE 99.—COST OF CHEMICAL PRECIPITATION AT PROVIDENCE, R. I.

Year	Pounds per 1,000,000 gal.		Water in sludge, per cent.	Cost per 1,000,000 gal.		
	Lime	Copperas		Chemical precipitation	Sludge disposal	Total
1903	606	65.0	94.63	$3.31	$2.44	$5.75
1904	683	58.0	92.46	3.42	2.57	5.99
1905	559	51.5	92.11	3.13	2.66	5.79
1906	638	72.1	92.58	3.50	3.10	6.60
1907	654	83.1	92.15	3.54	3.07	6.61
1908	726	....	91.77	3.42	3.43	6.85
1909	700	....	91.29	3.36	4.22	7.58
1910[2]	486	....	92.07	3.11	4.06	7.17
1911	438	....	92.57	....	....	....
1912	......	....	90.03	2.85[3]	2.49	5.34
1913	......	....	89.60	2.50[3]	2.30	4.80

[1] Since this chapter was prepared, lime treatment has been resumed.

[2] In 1910, the city began to experiment with the disinfection of its sewage, and the following year these experiments entailed so many modifications of operation that all the records could not be abridged into the usual summarized form. In 1912 chemical precipitation was abandoned and plain sedimentation followed by disinfection of the effluent was adopted. During the period when chemical precipitation was practised, the removal of suspended organic matter, as determined by albuminoid ammonia, was from 80.5 to 86.4 per cent., while during 1912 and 1913, it was 41.0 and 40.2 per cent. respectively. Lime treatment is now (1915) practiced again.

[3] These figures include the cost of disinfecting the sewage.

cost $235.71 annually and those for sludge disposal $4341.68, fuel charges were $669.90 and $19,213.36 was spent for chemicals. Repairs and renewals to machinery amounted to $858.44, to tanks $243.98, to buildings $510.01, to wharf $110.77; and to scow $1,074.89. The labor charge was $5,778.01 for chemical treatment and $12,010.97 for sludge disposal. The cost of lighting was $88.28 and $2,052.00 was spent for power. These figures, furnished by Bugbee, show an average annual operating cost of $47,188.01, or $6.99 per 1,000,000 gal. of sewage treated. The total cost of the precipitation plant up to Jan. 1, 1911, was $326,762.62.

London Practice.—The chemical treatment of the London sewage is substantially the same at Barking and Crossness; only the works at Barking will be mentioned here.

The precipitants used are lime and copperas. At a point 2250 ft. from the precipitation basins, a dose of lime is given to the sewage at the rate of 57.1 parts per 1,000,000 parts of sewage (475 lb. per 1,000,000 gal.). At a point 750 ft. from the basins a dose of copperas, 14.3 parts per 1,000,000 (119 lb. per 1,000,000 gal.) is added.

There are now (1915) 13 precipitation basins 30 ft. wide and 860 to 1200 ft. long. The basins have a depth of 8½ ft. at the effluent weirs, over which the clarified effluent passes in a sheet 4 to 11 in. thick, and thence through an outlet into the River Thames. When a basin is closed off for cleaning, its contents are allowed to stand quiescent for 2 hours, and then the top 4 ft. of supernatant liquid is drawn off through telescopic weirs. The remaining contents of the basin are discharged into a sludge sump, and the heavy stuff on the floor is pushed along by hand. The sludge is screened in passing to the sump in order to take out rags, sticks and other materials liable to interfere with the pumps which force the sludge to tanks. There it is allowed to settle for 24 hours, when the top liquid is drawn off by telescopic weirs, dosed with 285 parts (2370 lb. per 1,000,000 gal.) of lime and 143 parts (1190 lb. per 1,000,000 gal.) of copperas per 1,000,000 parts of sewage, and pumped back to the outfall sewer, to retrace its course through the plant. The settled sludge is taken by gravity from the tanks to what is called the sludge store, whence it is pumped directly into the sludge steamers for disposal at sea or to elevated sludge reservoirs.

About 348,000,000 U.S. gal. of sewage were treated daily on both sides of the river in 1911, in which work about $122,000 was spent for chemicals and supplies and about $24,000 for labor, making a total of about $146,000. This amounts to about $1.15 per 1,000,000 U. S. gal. It has been proposed to abandon the use of chemicals and to clean out the precipitation basins more frequently, as a result of investigations made in 1911.

The sludge contains from 91.95 to 92.28 per cent. of moisture and

amounts to about 9700 to 10,300 gal. per 1,000,000 gal. of sewage, according to the Fifth Report of the Royal Commission on Sewage Disposal. The average cost of disposal of the sludge, including interest and sinking-fund charges, was about 9.1 cts. per long ton. This is about $2.07 per 1,000,000 U. S. gal. treated. The total cost for treatment and sludge disposal comes to about $3.22 per 1,000,000 U. S. gal. received at the disposal works.

MIXING CHEMICALS WITH SEWAGE

In order to obtain the best results from chemical precipitation it is desirable to have a good mixture of the precipitants with the sewage before the latter enters the tanks. It is not advisable to add precipitants before pumping the sewage, because there seem good grounds for the opinion that the flocs formed by the precipitant are broken up and charged with more or less air, which buoys them up and interferes with their sedimentation. If the flocs are disturbed they seem to settle less uniformly and steadily than when they subside without interruption through slowly moving liquids.

One of the most satisfactory means of mixing the chemical dose with the sewage is a fish ladder (Fig. 110).

In a few cases where industrial wastes were treated, it was found impracticable to cause a proper mixing of the precipitants by ordinary methods. Accordingly, compressed air was allowed to escape for a time from perforated pipes in the bottom of the tank. This mixed the precipitant with the sewage very thoroughly and quickly, but could be practised only where quiescent settlement was followed.

The number and capacity of the tanks have a decided influence in a well-managed plant on the quality of the effluent, if the influent is subject to large surges of industrial wastes. Where this happens there is a great difference in the character of the sewage received at different hours. It is often found that the most satisfactory results are obtained by allowing the sewage to become of uniform quality by mixing the discharges at different hours in the tanks, although occasionally, when a special class of industrial waste is received daily at the same hour, it may be advisable to divert it into a tank for independent treatment. What actually happens to the sewage in an industrial town was described as follows in the report for 1894 of the Superintendent of Sewers of Worcester:

"The sewage at frequent intervals contains a large amount of sulphate of iron, commonly known as 'doses.' These come without any appreciable warning, frequently as many as six of them a day, and as the water is generally flowing at a speed of about 3 ft. per second, it can readily be seen that they may proceed some distance unnoticed. Now if this acid sewage gets into

a basin without sufficient lime having been mixed with it, the water in the entire basin, and, indeed, in all the basins it passes through, is turned a brown color. This may happen although the sewage 'goes acid' but a very few minutes, and the most careful watch must be kept to avoid this result which, though not materially affecting the quality of the effluent from a chemical or sanitary standpoint, is apt to bring the plant into disrepute in the thoughts of our many visitors who know very little of the science of sewage treatment.

"Not only does the sulphate of iron in the sewage bring about this result, but there are a number of 'doses' of dyestuff, tannery refuse and other industrial wastes which will act in a similar way. In the case of iron solutions, lime is sufficient to counteract this evil if applied in time, but with some of the other wastes it has scarcely any effect. In order to meet the demand, it is necessary to neutralize one dose with another, which, with the addition of the proper amount of lime, will effect a cure for color. If the different chemicals would always come at the same time and in proper proportions, this would be a very simple matter, but this is not the case. Either chemicals must be added to bring about this result or one 'dose' must be stored up to be used to offset another. On the ground of economy, the latter is the one thus far adopted."

Detention Period.—The period that the sewage must remain in the tanks after the chemical dose has been added to it depends on such factors as the amount and character of the suspended matter in it, the amount and nature of the trade wastes, the treatment of the effluent after leaving the tanks, and the operation of the tanks by the quiescent or continuous method, in parallel or in series.

The British experience is summed up in the Fifth Report of the Royal Commission on Sewage Disposal as calling for a detention period of 8 hours with continuous operation and 2 hours with quiescent operation, with sewage of a normal domestic character and average strength.

At Worcester, Mass., experience has shown that with the sewage of the class treated chemically there, the best results are obtained by sending the sewage first through roughing tanks and then through finishing tanks, the total detention period being 7 hours. The roughing tanks must be emptied every 2 to 4 weeks, and the finishing tanks every 3 to 6 weeks. The roughing tanks are usually operated in series and the finishing tanks in parallel. Operation in series results in removal of heavier and coarser matters in the roughing tanks with a tendency toward a thinner sludge in the finishing tanks.

VOLUME OF SLUDGE PRODUCED

The volume of sludge produced depends so greatly on the percentage of water it contains that perhaps the most satisfactory method of discussing the subject is by means of the dry solids in the sludge, reduced to the number of pounds daily per 1000 population. By adopting this method the

influence of differences in the water consumption and the ground-water leakage is eliminated, but a chance for serious error is introduced if the population of a city is only partially served by sewers and there are large storm overflows. Even in cities like Worcester and Providence, which are well sewered, it is probable that 10 per cent. of the people live in suburban streets where sewers have not yet been laid.

The volume of wet sludge with any percentage of water can be determined from the weight of dry solids in it by means of Table 106. The difference in the specific gravity of the dry solids makes comparatively little difference in the volume of sludge, as the table shows.

The quantity of solids per 1000 population produced by chemical precipitation at Worcester, Mass., cannot be obtained directly from the official figures, because the strong day sewage is treated by sedimentation and filtration and only the weaker night sewage by chemicals. Nevertheless a fair approximate estimate can be made by the method followed in Table 100. The detention period of the sewage in the tanks

TABLE 100.—SLUDGE DATA, WORCESTER, MASS.

	1909	1910	1911	1912
Sewage treated chemically:				
Million gallons sewage annually	3,686	3,580	3,610	4,201
Suspended albuminoid ammonia,[1] parts per 100,000	0.368	0.378	0.407	0.392
Wet sludge, gallons per 1,000,000 gal	4,210	4,450	4,949	4,551
Wet sludge, percentage of water	93.07	91.80	92.48	92.46
Total solids removed, tons annually	3,642	4,182	4,488	5,065
Solids removed per 1,000,000 gal., tons	1.00	1.17	1.24	1.20
Cost of chem. prec. per 1,000,000 gal	$5.12	$5.25	$5.53	$4.90
" " sludge disp., per 1,000,000 gal	4.28	4.53	4.60	5.31
" " chemical treatment, per 1,000,000 gal	9.40	9.78	10.13	10.21
Sewage settled and filtered:				
Million gallons annually	1,558	1,722	1,792	1,560
Suspended albuminoid ammonia,[2] parts per 100,000	0.739	0.763	0.734	0.789
Wet sludge, gallons per 1,000,000 gal	3,720	3,980	3,670	3,750
Wet sludge, percentage of water	95.71	....	95.6	95.06
Total solids removed, tons annually	1,036	1,071	1,206	1,205
Cost of treatment, per 1,000,000 gal	$9.78	$8.65	$7.49	$8.02
Total solids removed had treatment equalled chem. prec. in efficiency, tons per 1,000,000 gal.	2.06	2.36	2.24	2.41
Total solids, assuming precipitation, tons annually	3,209	4,064	4,014	3,760
Total solids:				
Tons annually, both processes	4,678	5,253	5,694	6,270
Tons, annually, had all been treated chemically	6,851	8,246	8,502	8,825
Estimated population	142,000	145,986[3]	150,000	154,000
Pounds per 1000 pop. daily	180	196	204	223
Pounds by chemical precipitation alone	140	156	160	180
Pounds by chemical precipitation assuming all sewage treated chemically and percentage of removal of suspended matter uniform during 24 hours.	264	309	310	314

[1] In sewage treated chemically. [2] In sewage settled and filtered. [3] U. S. Census return.

with chemical precipitation was 7 hours, while with plain sedimentation the sewage was given ½ hour in a tank 166.7 × 40 × 7 ft. deep. The quantities of calcium hydrate and carbonate in the chemical sludge cannot be estimated.

The quantities of dry solids in the sludge produced by chemical precipitation and plain sedimentation at Providence, R. I., stated in pounds per 1000 inhabitants daily, are given in Table 101.

The quantity of solids in the London sludge in 1903–6 averaged about 236 lb. daily per 1000 population in territory drained.

Speaking in general terms, with a given sewage, the greater the quantity of applied precipitants, the greater the quantity of sludge, and with two different sewages, a given quantity of precipitants will produce more sludge from the stronger sewage. The influence of screening and sedimentation in grit chambers on the results of chemical precipitation must also be taken into consideration.

TABLE 101.—SLUDGE DATA, PROVIDENCE, R. I.

Year	Treatment	Total solids daily, tons	Estimated population	Solids, lb. per 1000 pop. daily	Year	Treatment	Total solids daily, tons	Estimated population	Solids, lb. per 1000 pop. daily
1903	Lime and copperas.	21.4	188,500	228	1908	Lime.....	24.8	213,000	232
1904	Lime and copperas.	24.9	193,000	258	1909	Lime.....	28.5	217,000	263
1905	Lime and copperas.	22.3	198,600	225	1910	Lime.....	22.7	224,300	202
1906	Lime and copperas.	27.0	203,000	266	1912	Sedimentation...	16.5	235,600	140
1907	Lime and copperas.	27.3	208,000	262	1913	Sedimentation...	16.3	240,600	136

The figures for 1911 are incomplete owing to the changes made in the operation of the works during experimental investigations.

PRACTICAL SUGGESTIONS ON DESIGN OF PRECIPITATION TANKS[1]

The depth of the tank should be between 6 and 9 ft. Since the tanks should be cleaned very frequently, it is unnecessary to provide space for storage of sludge at the bottom of the tank, as was done in some of the older precipitation plants.

The length of tanks was formerly made about two and one-half times the width in England, but it has been found that better results were obtained with a greater proportional length. Tanks in operation in 8

[1] These notes were prepared by Julius W. Bugbee, superintendent of the Providence sewage treatment works and formerly chemist of the Worcester treatment works. They supplement the information in Chapters X and XI.

German cities varied from 5 : 1 to 16 : 1. Of the precipitation tanks in this country, those at Worcester have a ratio of 4.17 : 1, while the ratio at Providence is 2 : 1.

The bottom of the tank should rise toward the outlet and also slope from each side wall toward a drain running longitudinally through the tank. If the bottom could be given a grade of at least 10 per cent., the sludge would all flow to the drain. This is obviously impossible with large tanks, in which it is necessary to force the sludge adhering to the bottom into the drain by means of wooden scrapers. Hence narrow tanks, involving the least travel for the men employed in this work, are most economical in operation.

The inlets for sewage should be as near the full width of the tank as possible, in order to reduce the entrance velocity to the minimum and to avoid stagnant corners where septic action is likely to occur in a short time. For the same reason no dead ends should exist on the inlet and effluent channels, and the bottoms of these channels should be constructed to retard the sedimentation of suspended matters, just as is done in sewers carrying variable quantities of sewage.

The sluice gates through which the sludge is drawn off should be at the inlet ends of the tanks where most of the sludge will be deposited.

There should be a floating arm and valve for drawing off the supernatant liquid before removing sludge from a tank. The valve should be at the extreme end of the tank in order to remove the greatest possible quantity of water from the sludge.

Provision should be made for supplementing the work of the scrapers used in sludge removal by means of gates in the side walls, through which supernatant liquid may be introduced from adjoining full tanks for flushing.

The size of the individual tanks should be so proportioned to the flow as to give at least 4 units, in order that the tanks may be cleaned as often as once a week in hot weather, without reducing the desirable working capacity of the plant. Frequency of cleaning is an important operating duty, because of its value in preventing septic conditions in the effluent and also because of its effect on the work of sludge-pressing.

CHAPTER XIII

SLUDGE

The term sludge is generally used to designate the deposit which accumulates during the tank treatment of sewage by chemical precipitation, sedimentation, septic or hydrolytic tank process, and also the deposit from the final sedimentation of the effluent from trickling filters. Grit-chamber sediment and screenings are not usually classed as sludge, but when no preliminary separation of these solids is made, the sludge contains both of these elements. The quantity and character of sludge vary greatly, and for this reason it is difficult to estimate accurately sludge data for different places. Nevertheless, when treating similar sewages under similar conditions the quantities of sludge produced by different processes are in the following order of magnitude: chemical precipitation, sedimentation in plain, septic, Travis and Imhoff tanks. Experiments by Eddy and Fales at Worcester, made on a large scale under practical working conditions during 1902 and 1903, showed that the quantity of suspended solids in the effluent and sludge, and the volume of sludge from chemical precipitation, plain sedimentation and septic tank treatment averaged the amounts given in Table 102. (*Jour. Assn. Eng. Socs.*, vol. xxxvii, 1906, page 109.)

TABLE 102.—SLUDGE OBTAINED FROM WORCESTER, MASS., SEWAGE BY DIFFERENT TANK TREATMENTS, 1902 AND 1903

	Tons of dry suspended matter per 1,000,000 gal. sewage			Sludge, gal. per 1,000,000 gal. sewage
	Effluent	Sludge	Total	
Sewage	1.247	....	1.247	....
Precipitation[1]	0.250	1.435	1.735	4872
Sedimentation[1]	0.601	0.580	1.181	2544
Septic tank	0.840	0.161	1.001	548

[1] See also Table 100, page 464.

Most statistics relating to the quantity of sludge produced are reported in terms of volume of sludge per volume of sewage treated. The unit volumes of sludge so reported vary with the rate of sewage flow during the period covered by the observations. The information

would be much more satisfactory if reported in units per capita per given period of time, as in the last column of Table 103. The quantity of sludge produced at the sewage treatment works of a Massachusetts city of about 12,000 population, producing no industrial wastes worthy of consideration, has been recorded as the tanks have been cleaned. The treatment consists simply of passing the sewage slowly through horizontal tanks which are cleaned every 3 weeks. The quantity of sludge produced in 1912 is reported in Table 103.

TABLE 103.—QUANTITY OF SLUDGE PRODUCED AT A MASSACHUSETTS CITY, 1912

Period	Flow of sewage, gal. per cap. per day	Quantity of sludge		
		Gal. per 1,000,000 gal. sewage	Cu. yd. per 1,000,000 gal. sewage	Cu. ft. sludge per cap. per day
Maximum, Oct. 4–Oct. 25	29.2	4271	21.2	0.0165
Minimum, May 10–May 31	112.6	908	4.5	0.0146
Average, Nov. 7, 1911–Nov. 22, 1912	64.1	1822	9.0	0.0162

Character and Composition of Sludge.—The sludge from plain sedimentation is gray in color and in most cases possesses a very offensive odor. An exception is the sludge at Providence, R. I., resulting from sedimentation of disinfected sewage, which, when observed in the summer of 1912, did not possess a particularly offensive odor. The sludge from sedimentation tanks is slimy, usually quite dense, and very difficult to press. It can be dried by spreading it upon porous beds, but in order to dry it as quickly as possible and prevent the continuance of objectionable conditions about the beds, it should be applied in a very thin layer to enable the water to drain rapidly away.

The sludge from chemical precipitation is greater in volume and contains a greater weight of solid matter per 1,000,000 gallons of sewage treated than any other sludge. The large volume is due to precipitates of chemicals introduced into the sewage. Left in the tank, it undergoes decomposition, like that from sedimentation in a septic tank, but at a much slower rate. Gas is produced in substantial quantities, and the density of the sludge is increased by standing. If such sludge contains considerable iron the surface may be red, due to oxidation, but below the surface the sludge is generally black. The odors arising from it, while objectionable, are by no means as offensive as those from sludge from sedimentation tanks. While precipitation sludge may be described as slimy, it is also gelatinous on account of the hydrate of iron or

hydrate of alumina in it. When spread upon porous draining beds the water will drain away or evaporate, leaving after several weeks a stratum of slimy sludge of about the consistency of lard, containing 70 to 80 per cent. of water.

Septic tank sludge is black and usually possesses a very offensive odor, due largely to hydrogen sulphide gas. The sludge produced by the septic process at Birmingham, England, is said to possess relatively little odor because of the copper compounds in it. Sludges produced at some places have been reported as possessing relatively little odor, due in most cases to long digestion in the septic tanks. The coarser particles settling in the tanks are largely disintegrated by the putrefactive action going on in the sludge, so that it contains relatively small quantities of coarse material. The sludge can be dried on porous beds, if spread out in thin layers, but objectionable odors are to be expected.

The sludge from the Imhoff tank is quite different in character from that resulting from other processes of treatment, in that it contains large quantities of gas. Although usually containing a larger proportion of solid matter than other sludges, it flows readily because of the fluidity imparted by the entrained gases. It is less offensive than either of those previously described, the odor possessed by it being like that of hot tar or sealing wax. When drawn off on to porous beds in layers 6 to 10 in. deep, the solids are carried to the surface by the entrained gases, leaving a stratum of relatively clear water below on the bed, which drains rapidly away. As the sludge dries, the gases escape, leaving it more or less spongy or porous.

The sedimentation of trickling filter effluents gives a sludge quite different from any produced by primary tank processes. It is brownish, relatively inoffensive when fresh, flocculent, and usually contains a very high percentage of water. It undergoes decomposition more slowly than other undigested sludges, although when it contains many worms it may become offensive quickly. In warm weather gases are liberated from it, and large masses of it may at times be carried to the surface by entrained gas. It is the most difficult sludge to dry on porous beds. At Worcester, where it contained large quantities of hydrate of iron, it was dried with extreme difficulty except in very thin layers. At Fitchburg, Mass., sludge from the secondary settling basins is pumped into the influent to the Imhoff tanks in order to mix it with the Imhoff sludge, and thus take advantage of the gases and fibrous matter of the latter to produce a porous and quickly draining sludge. By this method any unstable organic matter in the secondary tank sludge, such as worms, will be put through the rotting process to reduce it to a stable condition, in which it will not be likely to decompose and give off offensive odors when exposed on the sludge-drying beds.

There is a striking similarity in the proportion of organic matter, as

represented by volatile solids, in all sludges recorded in Table 104. The large amount of sulphate of iron in Worcester sewage causes the presence of varying quantities of iron in the several sludges. A large quantity of iron is precipitated from the sewage as sulphide in the septic tank, and the oxidation of the iron causes its precipitation in the trickling filter, as is apparent from the very high percentage of iron in the sludge from the secondary sedimentation basins. The iron calculated as oxide of iron (Fe_2O_3) constitutes about 54 per cent. of the mineral matter contained in this sludge.

TABLE 104.—ANALYSES OF DRIED SLUDGES FROM CHEMICAL PRECIPITATION, PLAIN SEDIMENTATION, SEPTIC TANK TREATMENT, IMHOFF TANK AND SECONDARY SEDIMENTATION TANKS FOR TRICKLING FILTER EFFLUENTS, AT WORCESTER, MASS.

	Chemical precipitation, per cent.	Plain sedimentation, per cent.	Septic tank, per cent.	Imhoff tank, per cent.	Sec. sed. tank, per cent.
Volatile solids	47.26	51.04	43.94	49.12	51.05
Fixed solids	52.74	48.96	56.06	50.88	48.95
Silica (SiO_2)	25.46	28.59	20.41	....	....
Iron sulphide (FeS)	0.02	0.57	16.58	....	....
Iron, not as sulphide	5.80	2.45	2.98	....	....
Total iron	....	....	....	4.78	18.57
Sulphur, not as sulphide	0.44	0.60	0.64	....	....
Aluminum oxide (Al_2O_3)	0.57	1.94	7.29	....	....
Calcium oxide (CaO)	2.88	0.61	1.14	....	....
Magnesium oxide (MgO)	0.74	0.29	0.97	....	....
Phosphorus pentoxide (P_2O_5)	0.47	1.71	1.85	....	....
Carbon (C)	28.60	31.26	23.95	....	....
Hydrogen (H)	4.21	4.46	3.64	....	....
Nitrogen (N)	2.77	3.05	3.01	2.63	2.97

Table 105 gives the percentage of water and the relation of mineral to organic matter in sludges from various places and different methods of treatment. The Philadelphia results might have been different had a deeper tank been used, as the tank there used was much less deep than those designed by Imhoff.

Measurement of Sludge.—The method followed at the Columbus Sewage Experiment Station is thus described:

"In measuring the sludge deposit in a tank from which the sewage had been drained, the depth of the layer was measured in 11 lateral sections, as already stated. At each point where a measurement was made, a rep-

resentative sample was collected. The results of these measurements were averaged, and the 11 representative portions collected were mixed for analysis.

TABLE 105.—COMPOSITION OF SLUDGE
(Compiled from "Sludge Disposal" by Kenneth Allen)

Place	Character of sludge	Wet sludge				Dried material			
		Weight, lb. per cu. yd.	Specific gravity	Water contained per cent.	Dry material, per cent.	Mineral, per cent.	Organic, per cent.	Nitrogen, per cent.	Fats, per cent.
Germany:									
Frankfort-am-Main.	Plain sedimentation.	......	.....	91.07	8.93	43.0	57.0	2.85	.....
Frankfort-am-Main.	Precipitated by lime.	......	.....	90.85	9.15	54.6	45.4	.....	.....
Frankfort-am-Main.	Precipitated by sulphate of iron and lime.	......	.....	80.96	19.04	30.0	70.0	.....	.....
Stuttgart.......	Septic tank......	......	.....	77.3	22.7	66.9	33.1	.....	.....
Recklinghausen.	Imhoff tank......	......	.....	79.34	20.66	54.8	45.2	1.56	6.41
Essen..........	Imhoff tank......	......	.....	75.6	24.4	45.08	54.92	1.22	4.89
Bochum........	Imhoff tank......	......	.....	75.88	24.12	59.49	40.51	1.102	8.73
United States:									
Columbus, O...	Grit chamber....	1825	1.081	82.4	17.6	55.1	44.9	1.2	6.94
Philadelphia ...									
Tank 12......	Septic, screened sewage	......	1.036	90.0	10.0	51.0	49.0	1.3	8.1
Tank 13......	Septic raw sewage	......	1.053	86.1	13.9	52.0	48.0	1.4	7.4
Tank 17......	Septic raw sewage	......	1.043	87.7	12.3	50.0	50.0	1.3	7.2
Reading, Pa.....	Septic..........	......	.....	91.83	8.17	53.0	47.0	.....	.....
Columbus, O....	Septic..........								
Tank A.......	....	1836	1.089	83.3	16.7	73.8	26.2	1.5	5.6
Tank B.......	....	1823	1.080	82.3	17.7	75.2	24.8	1.5	6.0
Tank C.......	....	1823	1.080	83.2	16.8	74.4	25.6	1.4	6.3
Tank D.......	....	1800	1.069	84.7	15.3	73.2	26.8	1.2	8.9
Tank E.......	....	1833	1.087	83.7	16.3	69.3	30.7	1.1	7.2
Waterbury, Conn.									
Tank 2.......	....	1721	1.02	86.3	13.7	57.0	43.0	1.2	11.2
Tank 3.......	....	1738	1.03	85.4	14.6	52.0	48.0	1.5	10.4
Saratoga, N. Y..	Septic..........	......	1.025	94.0	6.0	25.0	75.0	.....	.....
Philadelphia....	Imhoff..........	......	1.085	82.5	17.5	62.0	38.0	1.2	6.5

"The apparatus used in measuring the deposit in a septic tank while in operation consisted of a glass tube 2.5 ft. long and 0.5 in. in diameter, open at both ends and fastened parallel to the side of a wooden rod 12 ft. long. Through the glass tube a fine wire was drawn, at the lower end of which was fastened a flexible rubber stopper, the smaller end uppermost. The wire extended up through the glass tube to the top of the wooden pole, being guided here and there by screw-eyes.

"In making a measurement the rod was lowered into the liquid in the

tank, and slowly inserted into the deposit on the bottom. After a sufficient time had been allowed for displacement in the tube the wire was pulled, drawing the rubber stopper into the lower end of the glass tube. The rod was drawn up, and the depth of the sludge in the tube measured." (Report on Sewage Purification, page 74.)

A similar apparatus used by Kinnicutt at Worcester was somewhat more satisfactory because of the larger size of the glass tube employed, about 7 ft. long and 2½ in. in diameter. A heavy wire was run through the tube and attached to a hollow rubber ball large enough to close the end of the tube. This apparatus gave fairly satisfactory results, even when the sludge contained considerable coarse material which would not flow into a tube 0.5 in. in diameter.

In the practical operation of tanks it is desirable to know the depth of sludge in the several basins at frequent intervals. An easy method of determining the position of the surface of the sludge is by slowly lowering a small tin dipper, attached to a pole, into the liquid until it has reached a depth assumed to be slightly above the sludge line. By

TABLE 106.—WEIGHT AND SPECIFIC GRAVITY OF 1 CU. YD. OF SLUDGE WITH DIFFERENT PERCENTAGES OF MOISTURE AND CONTAINING SOLIDS OF DIFFERENT WEIGHTS PER CUBIC FOOT

Percentage of moisture in sludge	Weight of dry residue in pounds per cubic foot														
	68			78			92			110			134		
	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent. water	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent. water	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent. water	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent. water.	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent. water	Weight per cu. yd., lb.
98	1688	5.04	1.002	1692	5.08	1.004	1696	5.13	1.007	1699	5.18	1.008	1702	5.23	1.010
97	1689	3.35	1.003	1695	3.38	1.007	1702	3 41	1.010	1707	3.44	1.013	1711	3.47	1.016
96	1691	2.51	1.004	1699	2.53	1.009	1708	2.55	1.013	1715	2 57	1.018	1721	2.59	1.022
95	1692	2.01	1.004	1703	2.02	1.010	1713	2.03	1.017	1723	2.04	1.023	1731	2.06	1.027
94	1694	1.67	1.005	1706	1.68	1.012	1718	1.69	1.019	1730	1.70	1.027	1740	1.71	1.033
93	1695	1.43	1.007	1710	1.44	1.015	1724	1.44	1.023	1738	1.45	1.032	1750	1.45	1.039
92	1697	1.25	1.007	1713	1.25	1.017	1730	1.26	1.027	1745	1.26	1.035	1760	1.26	1.044
91	1698	1.11	1.008	1717	1.11	1.019	1735	1.11	1.030	1752	1.11	1.040	1770	1.11	1.050
90	1700	1.00	1.009	1720	1.00	1.020	1740	1.00	1.032	1760	1.00	1.044	1780	1.00	1.056
89	1701	0.91	1.010	1724	0.91	1.023	1748	0.91	1.037	1768	0.91	1.047	1790	0.91	1.062
88	1703	0.83	1.010	1727	0.83	1.025	1752	0.83	1.040	1776	0.83	1.054	1800	0.83	1.068
87	1704	0.77	1.011	1730	0.77	1.027	1758	0.77	1.043	1785	0.76	1.059	1810	0.76	1.074
86	1706	0.71	1.012	1734	0.71	1.029	1764	0.71	1.046	1793	0.70	1.064	1820	0.70	1.080
85	1707	0.66	1.013	1737	0.66	1.031	1770	0.66	1.050	1802	0.65	1.069	1831	0.65	1.086
84	1709	0.62	1.014	1741	0.62	1.033	1776	0.62	1.054	1810	0.61	1.074	1841	0.61	1.093
83	1710	0.59	1.015	1744	0.58	1.035	1782	0.58	1.058	1818	0.57	1.079	1852	0.56	1.099
82	1711	0.56	1.016	1748	0.55	1.037	1788	0.55	1.062	1827	0.54	1.084	1863	0.53	1.106
81	1713	0 53	1.017	1751	0.52	1.039	1794	0.52	1.065	1835	0.51	1.089	1874	0.50	1.112
80	1714	0.50	1.018	1755	0.49	1.041	1800	0.49	1.068	1844	0.48	1.094	1885	0.47	1.118
79	1715	0.47	1.019	1758	0.46	1.043	1807	0.46	1.072	1853	0.45	1.100	1896	0.44	1.125
78	1717	0 45	1.019	1762	0.44	1.045	1813	0.44	1.076	1863	0.43	1.106	1908	0.42	1.132

repeating the test at progressively increased depths, each time withdrawing the dipper carefully so that the collected sludge, if any, will remain in it, the operator can judge from the graduations on the pole at what depth he first encounters the sludge layer. A method used with Imhoff tanks was explained on page 443.

Volume and Weight of Sludge.—The physical properties of sludge considered in estimating the size of the sludge chamber of an Imhoff tank have been given in Table 93 and Fig. 104. A more convenient method of stating the weight, volume and specific gravity for some purposes is followed in Table 106.

REMOVAL OF SLUDGE FROM TANKS

The methods of removal may be divided into 2 distinct classes depending upon whether the tank is thrown out of commission and the liquid portion withdrawn preparatory to removing the sludge, or the sludge is removed while the tank is in operation without withdrawing the supernatant liquid. In the second case mechanical appliances or steeply sloping sides are necessary to concentrate the sludge near the outlets.

Required Slope of Bottom of Tank.—The slope of the floor of the tank should be governed by the method of operation to be followed. If the supernatant liquid is to be drawn off before removing the sludge and some scraping and flushing are permitted, the floor should have a slope which can be readily cleaned by scraping and will permit thin sludge to flow toward the drains without much assistance. It is not practicable for laborers to stand upon a sludge-covered floor which has a slope much greater than 1 in 20, on account of their tendency to slip. In many instances, particularly with large tanks, it is impracticable to provide means for carrying the sludge to the outlet by gravity. Schmeitzner says ("Clarification of Sewage," 1910) that at Frankfort the tanks have a serrated longitudinal section with slopes of 1:10, the bottom of the tanks being covered with glazed tile. Even with this slope, water under pressure is sometimes required for complete cleaning. Elsner says:

> "The attempt is, therefore, made to so construct the bottom of the tanks that the sludge in pumping will always flow by gravity to the pump well. The slope in general use, say 1:100 (Mannheim and Cassel) to 1:45 (Hannover), is not sufficient, for experience has shown that some aid by manual labor cannot be dispensed with in these tanks. For an easy automatic flow with settled sludge containing at least 90 per cent. of water, a slope of 1:10 to 1:15 is necessary, depending on whether there is much sand and coarse material, or whether there is a fine fluid sludge." ("Treatment and Utilization of Sludge," 1912, page 31.)

In general, in tanks of considerable size a series of sumps with steeply

sloping sides is necessary for complete removal by gravity. Such construction is expensive and at many plants, including most of those in the United States, the bottoms of the tanks have only a slight slope toward the outlet and hand cleaning is resorted to for complete removal of sludge.

The basins at Worcester are about 7 ft. deep, and each is provided with a central drain running its entire length, having a slope of 1:80. The floors of the old basins slope from the side walls to the central drains at the rate of 1:33⅓ and those of the new basins at the rate of 1:20. On an average, the sludge is removed from these basins when it has accumulated to a depth of about 2 ft., equivalent to 100,000 gal. per basin. When the gates are first opened the sludge will flow by gravity, but as it is drawn down it is necessary to send men in to scrape it toward the central drains, Fig. 111, and also to add sewage to reduce its density and flush it along the floor and drains. The labor of 1 man for about half a day and about 15,000 gal. of sewage for flushing purposes, are required for removing 100,000 gal. of sludge resulting from the chemical precipitation processes. This sludge, including sewage used for flushing, contains about 93.5 per cent. water.

During recent years, part of the raw sewage has been passed through a settling basin and discharged onto filter beds, the period of sedimentation being about half an hour. This sludge differs materially from that resulting from chemical precipitation and is much more difficult to handle. The labor of 1 man for 1 day, and from 50,000 to 75,000 gal. of sewage, are required for the removal of 100,000 gal. of this sludge which, including sewage used for flushing, contains about 95.6 per cent. water.

The quantity of sewage passed through the chemical precipitation plant and the quantity passed through the sedimentation tank in 1911, together with the respective quantities of sludge produced, the time required for removing it from the basins and the cost of labor, are given in Table 107.

Table 107.—Sludge Removal Costs at Worcester, Mass., 1911

Treatment	Quantity of sewage in million gallons	Quantity of sludge in million gallons	Days labor required cleaning	Cost of labor at $2 per 8-hr. day	Cost of labor per 1,000,000 gal. sewage
Chemical precipitation..	3610	17.866	89.33	$178.66	$0.049
Sedimentation.........	1792	6.575	65.75	131.50	0.073

In considering the relative costs of handling the sludge from these two processes, it should be borne in mind that the heaviest of the sewage, coming during the day time, was treated by sedimentation, and that the sewage receiving chemical treatment was largely that flowing at night,

FIG. 111.—Cleaning a precipitation basin at Worcester, Mass.

which was considerably weaker than the day sewage. From these figures, it appears that no very large expenditure is justified for mechanical appliances or a more expensive type of basins, simply for the purpose of avoiding the cost of hand cleaning.

Perhaps a more important element in the cost of hand cleaning is the length of time during which it is necessary to keep the basins out of use. For several hours before cleaning begins, the basins must be cut out to allow for the sedimentation of the solid matter in the supernatant sewage. After this has been completed considerable time is required for drawing off the supernatant liquid before sludge removal can be begun. It is estimated that at Worcester 10 per cent. of the chemical precipitation basin plant is out of use on account of sludge removal and that about the same proportion of a similar plant for the sedimentation of raw sewage would be out of use for the same purpose.

These disadvantages, while obviated by the use of tanks so designed that the sludge can be drawn off during operation, in a measure are often offset by the more dilute sludge drawn from tanks of the latter type, as pointed out by Schmeitzner ("Clarification of Sewage," page 19).

Slope of Floor and Drains Required for Convenient Removal of Sludge from Secondary Sedimentation Basins.—C. B. Hoover states that two secondary sedimentation basins at the sewage treatment works at Columbus, Ohio, were originally built with concrete bottoms and drains. The slope given to the latter was 1:125, while some of the floor slopes were as small as 1:400, and 95 per cent. of the floor had a slope less than 1:100. At the time these basins were constructed, the opinion seemed to prevail that the sludge which would be deposited in them would not decompose readily, and consequently infrequent cleaning would make it permissible to sacrifice floor slopes in order to increase the capacity of the basins. It is stated, also, that there were other reasons not of an engineering nature, partly responsible for building the basin floors with such flat slopes.

It was found necessary, however, to clean the basins at intervals of 4 to 6 weeks by scraping and flushing. In 1912 the original concrete bottom in 1 basin was removed and a new floor built, having a minimum slope of 1:50 to the drains, which were built with a slope of 1:125. The drains were also so arranged as to shorten the distance the sludge would have to flow, and increased flushing facilities were provided.

Sludge Removal by Gravity.—With respect to removal by gravity, Elsner says:

"A favorable concentration of the sludge at the bottom should be aimed at in the design. This end is most frequently attained with wells. As already mentioned, these are almost universally arranged for the removal of the sludge without preliminary emptying. With their comparatively

small dimensions it is usually easy to give the bottom such an inclination that the sludge will flow by itself toward the suction pipe of the sludge pump at the center. A slope of 2 vertical on 1 horizontal, as is found in the so-called Dortmund tank, and which is also used in England, suffices for all cases. A slope of less than 45 deg., as in the sludge-well constructed in the clarification tank at Frankfort, will permit a slippery sludge to slide off if submerged. The angles between the vertical walls and the conical base should receive especial attention, as experience indicates that the sediment in the sludge settles here. This can be prevented to a certain extent by rounding these corners.

"Naturally, the degree of roughness of the bottom helps determine the slope. In large plants it is well to make experiments with the sludge which comes from the sewage to be treated, unless the slopes have been determined by reliable experiments with different kinds of sludge on different surfaces from which it slips off by its own weight.

"It should furthermore be noted that in course of time a sticky coating is deposited on the smooth surfaces, reducing their efficiency very considerably—especially in the case of smooth enameled or glazed surfaces and those of glass—and that their cleaning necessitates a cessation of operation.

"The cone formed at the bottom of the wells corresponding to the natural slope of the earth will not suffice for a free removal of the sludge.

"It has been shown by experiments of Schoenfelder at Elberfeld that a steeper slope is required to secure an automatic sliding of the sludge if removed under water than if the supernatant liquid is first drawn off, and that special precautions should be taken in the process. Here it was observed that the sludge was deposited in horizontal layers not of uniform thickness, parallel to the bottom. When the sludge was drawn off at the deepest point, a funnel was formed. After this the sludge failed to slide, although having a slope of 1:3, but this did occur immediately after drawing off the supernatant sewage. The explanation of this is that the difference in weight between the saturated sludge and the turbid sewage above is too slight to overcome the friction of the surface at the bottom and of the surface in contact with the turbid sewage; for the weight of the sludge is reduced by that of the displaced sewage, while in the case of empty tanks the weight of the sludge becomes effective. The funnel mentioned gradually closed in again under water, so that in a half hour it was always smooth and horizontal.

"This was confirmed by experiments at Cologne. Here the sludge was to be pumped from under water out of sumps having a slope of 1:1. It soon appeared that after a few minutes only water came out, which found its way through the compact sludge near the suction strainer and carried with it only a few fragments of sludge which it was able to dislodge. This, which is also confirmed by practice and experiments elsewhere, proves that the removal of sludge under water and without stirring up the deposited material is only practicable before the sludge is firmly settled in place. The composition of the sludge is of importance in this connection as the greasy material forms a light but firm mass, while sludge from septic tanks which is

kept in motion by frequent partial removal, as in the Emscher (Imhoff) tank, and, by the gases rising from it, can easily be removed during operation.

"In these plants, which, as is known, are a combination of short sedimentation tanks with septic chambers below, the difference in quality between the fresh and septic sludge is taken into account by giving the floor of the upper part a slope of 1½:1, and in the most recent structures this is covered with glass plates laid on reinforced concrete supports to lessen the friction. Such precautions are necessary in order to induce the settled sludge to slide down in thin layers to the lower chamber as soon as posssible.

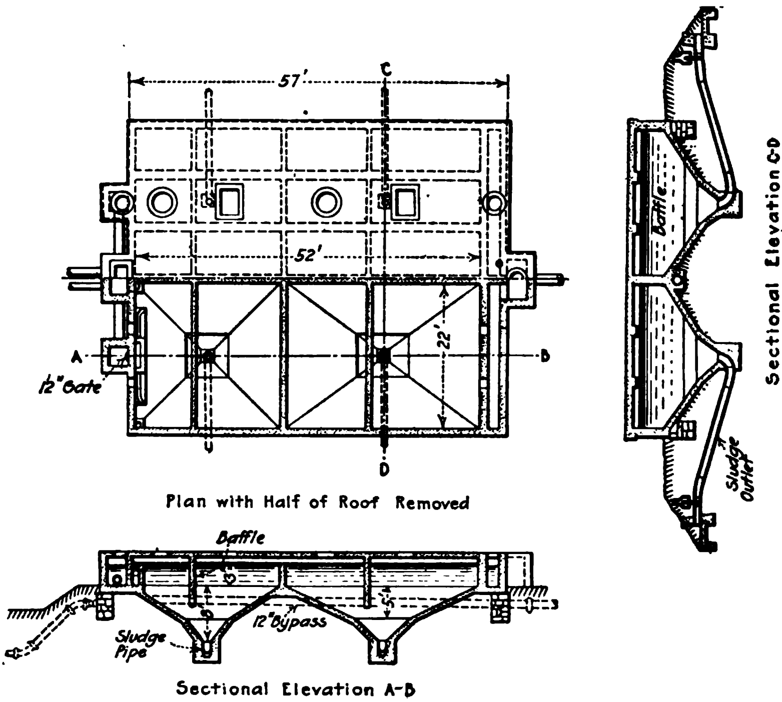


FIG. 112.—Settling tanks at North Attleboro, Mass.

"As the experiments of Grimm (which led him to introduce sedimentation plates in the tanks) have shown, this is prompted by the fact that colloidal matter has a marked tendency to form a gelatinous coating by friction, or even by contact with a solid body. This is then set in motion on the steep surface by gravity, and in rolling down carries with it the particles of sudge which are in the way. In order to convey the septic sludge, which fills the lower tank in a great mass, to the sludge pipe, a slope of 1:2 is sufficient, to which may be added a flushing pipe to be described later." ("Treatment and Utilization of Sludge," 1912, page 37.)

In some of the more recent American plants provision is made for the withdrawal of sludge by gravity and during operation of the tank. The tanks at North Attleboro, Mass., Fig. 112, designed by Barbour, are examples.

At Gloversville, N. Y., a city of about 20,000 persons, in which there are some 25 tanneries, the authors designed both primary and secondary sedimentation tanks of the Dortmund type.

The sludge collected in the primary tanks (Fig. 86) is of rather unusual character, because of the large quantity of tannery wastes entering the sewers, which contribute a great amount of suspended matter, and lime, alum and iron salts as well, so that the treatment is more like chemical precipitation than simple sedimentation. The sludge is generally heavy. These tanks have been in operation since 1912. Sludge is removed without drawing off the supernatant liquid. Some difficulty was experienced in drawing off the sludge from one of the tanks which finally had to be emptied. When empty the slope of the cone at the bottom of the vertical wooden inlet tube was increased to about 60 deg. Since then no difficulty has been experienced. The other tank has not been changed and the sludge appears to slide off the wooden cone and down the concrete hopper in a satisfactory manner. Hazen found in the tanks at the Columbian Exposition (Fig. 85) that even with a slope of 1.7 vertical to 1 horizontal, some sludge required pushing down to the lower part of the sump. This necessity was, however, partly due to laps on the iron plates of which the tanks were constructed.

At the Lawrence Experiment Station, the bottom of a vertical settling tank had a slope of about 17 per cent. This inclination was found to be much too flat, and it was changed to 60 deg. With the steeper slope it was found that the sludge would slide tc the outlet and could be blown out without drawing the tanks. (Report Mass. St. Bd. Health, 1911, page 278.)

"If the sludge is to be removed during the operation of the tank, the sludge sump at the inlet must be provided with steep sloping sides, at least 1:1, for otherwise the sludge will contain too much water. The bottom of the sump must be from 1.5 to 2.5 meters (4.92 to 8.20 ft.) deeper than that of the main tank and, in case of the gravity removal of the sludge, must slope also toward the sludge drain pipe or the outlet valve (page 64).

"Continuous operation possesses the great disadvantage that the sludge obtained contains a very large percentage of water. With a few exceptions, where the sludge is pumped to a sludge-disposal field, as in Mannheim and Birmingham, it is very important to obtain a sludge with as low a percentage of water as possible. The sludge produced under continuous operation of the tanks must be dried on special sludge areas, for the cost of drying will be very high if this is undertaken in sludge presses, centrifugal machines or by evaporation. This disadvantage appears to offset the advantage of continuous operation, which permits a longer time to elapse between

successive cleanings of the tanks. Of the newer more important plants in Germany, only in the settling tanks at Elberfeld-Barmen is the sludge removed while the tanks are in service." ("Clarification of Sewage," Schmeitzner, page 65.)

Elsner says in his "Treatment and Utilization of Sludge," 1912, regarding the removal of sludge during operation:

"Its principal advantage is in avoiding the costly removal of the turbid sewage, which in most places must be drawn off by pumps from tanks as well as from wells; and especially where the process necessitates frequent cleaning, this is an important consideration. Moreover, the entire plant can be in use, while otherwise to avoid overloading it must be constructed of greater size in order to allow for those parts which lie idle during cleaning. It can be so designed, moreover, that only the dried sludge is exposed, provided closed pipes are used and the drying is done by a mechanical process described later on, so that the demands of hygiene are more completely met and foul odors are almost eliminated. But even if sludge is dried in the open air this method offers great advantages, especially if the places for drying are at some distance from the treatment plant. A disadvantage in most plants cleaned during operation lies in the fact that their sludge contains a greater amount of water, not less than 95 per cent. Where it can be utilized in this wet condition without further transportation or where ample areas for drying with favorable sub-soil and location are available, this matter is of less importance" (page 36).

Mechanical Devices for Sludge Removal.—To obviate the necessity for hand cleaning, many mechanical appliances have been developed, particularly in England and Germany.

At Bolton, England, the Ashton-Booth sludge-pushing car is in use. This has a vertical steel framework with grooves in which 1-in. boards can be slipped; the bottom boards have rubber facings. The car runs on rails and spans the tank. It is operated by allowing sewage from an adjoining tank to fill up behind the boards in the frame until sufficient pressure is attained to push the sludge to the lower end of the tank, the floor of which has a slope of 1:80. When the lower end is reached, the car can be returned to its original position by producing a head against the other side of the board apron.

At Bremen a somewhat similar apparatus is used, but is drawn by a rope and windlass at the lower end of the tank. The tanks, which are 65.6 ft. wide, are divided into 4 compartments by timbers which project above the floor about 4 in., on which the wheels of the car run. For shifting the car from 1 compartment to the next, 4 wide rollers are used, which may be lowered by means of screws, thus lifting the car from the track and allowing it to be pushed by hand to the next track.

The Fidler sludge collector has been installed at Birmingham, Bury and other places in England, by Ham, Baker & Co. This consists of a

spiral band of sheet steel which is revolved by means of gears and gradually draws the sludge toward an outlet under the center of the band, as shown in Fig. 113. If the sewage is acid it may corrode the wire ties holding the arms of this sludge-remover, which led to the substitution of the Ashton-Booth sludge plow at Bolton for this apparatus.

Still a third type consists of a perforated pipe which is made to revolve around the center of the bottom of a circular tank. This perforated pipe is connected with the sludge outlet and the pressure of the over-lying liquid forces sludge into the pipe as it is revolved.

Devices for Drawing off Supernatant Liquid.—For removing the liquid preparatory to cleaning, several devices have been used. A swinging

Fig. 113.—Fidler sludge remover, Bolton, England.

arm supported by 1 or 2 floats, Fig. 84, and held so that the inlet is far enough below the surface to prevent the entrance of grease or surface scum, is used at Marlboro and Worcester, Mass., and at many European plants.

The apparatus shown in Fig. 114, designed by Prof. Robert Spurr Weston, has proved effective in handling both the supernatant liquid and the sludge, and is inexpensive.

A design recommended by Willcox & Raikes of London is shown in Fig. 95.

At some places in the United States valves at different elevations have been provided, and in many cases where small tanks are in use,

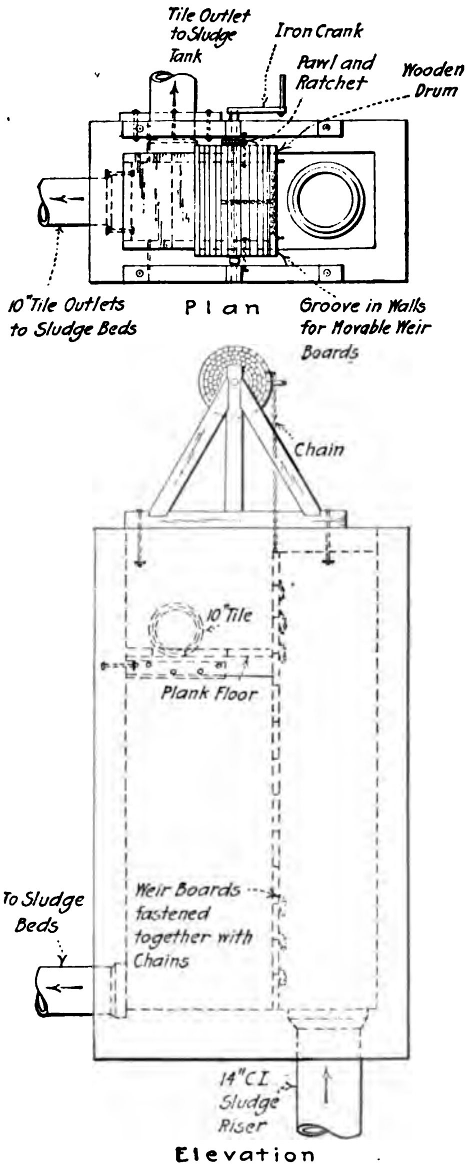


Fig. 114.—Movable weir for drawing off suspernatant liquid and sludge.

no provision is made for drawing off the supernatant liquid other than pumping it out with a portable outfit.

PUMPING SLUDGE

Sludge pumping is attended with obvious difficulties which must be overcome by special apparatus. Sludge may be raised by (1) compressed

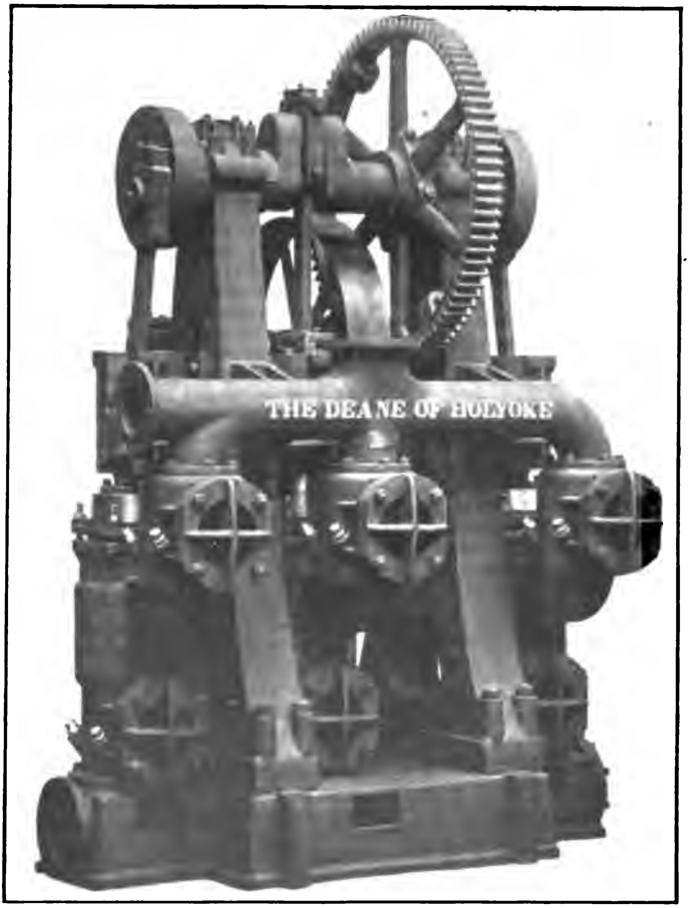


FIG. 115.—Stuff pump used for sludge at Worcester, Mass.

air ejectors, (2) direct-acting pumps, (3) centrifugal pumps, (4) membrane pumps, (5) air lift, and (6) vacuum pumps. The United States has but few examples of special apparatus for raising sludge. In many of

the smaller plants an ordinary hand diaphragm pump is used when necessary.

At Worcester, Mass., two methods are in use, one to raise the sludge from settling basins to the sludge storage tanks, the other to pump it into the filter presses. For the first purpose a Shone ejector is in use. This apparatus, described in Volume I, page 680, is also used at Providence, R. I., for pumping sludge. The air for this device was formerly compressed at Worcester by power generated by the tank effluent on its way to the

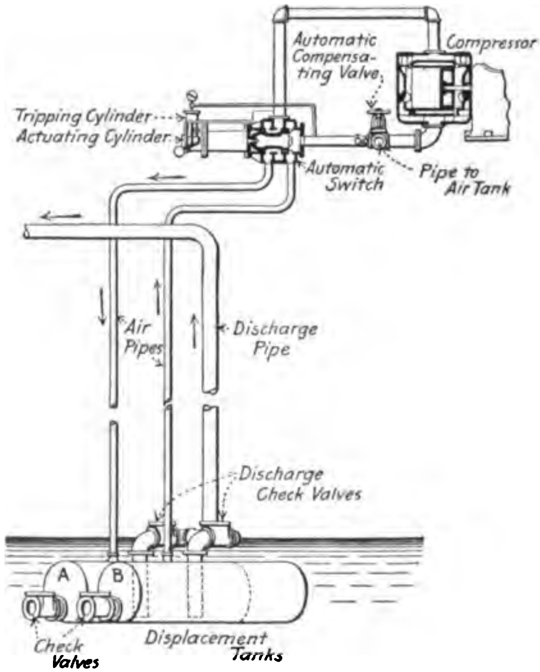


FIG. 116.—Return-air displacement pumps (Ingersoll-Rand).

river. A steam-driven compressor was substituted for the belt-driven compressor in 1897 when the tank effluent was conveyed to filter beds. For filling the presses triplex stuff pumps, Fig. 115, were installed, such as are used in paper mills for pumping pulp. They are provided with bronze ball valves which provide a large waterway and are easily accessible for cleaning.

Centrifugal pumps are used at Baltimore, Md., to lift sludge from the sedimentation to the digestion tanks.

In Germany various forms of vacuum or compressed air sludge-lifting apparatus are used. An American example of such equipment is the Ingersoll-Rand "return-air" system of pumping, shown in Fig. 116.

Air is being removed from the receiving tank *B* by the compressor and is being compressed in tank *A*. Sludge is entering tank *B* through the inlet check valve and is being driven from tank *A* through the outlet check valve and discharge pipe. When tank *B* is completely filled an automatic switch reverses its position. Air at full pressure in tank *A* expands through the automatic switch, air tank and compressor, into the air pipe leading to tank *B*, until equilibrium is recovered at a higher pressure. Then the compressor takes up the load, forcing air from tank *A* through the automatic switch into tank *B*, displacing the sludge in the latter. The moment the pressure in tank *A* is reduced below the hydraulic head, the sludge will enter the tank and fill it, when the automatic switch will reverse itself.

Another form of sludge pump, employed at Hannover, Frankfort and Mannheim, Germany, is what may be termed the membrane type, using the same principle of design as the diaphragm pump described in Volume II, page 67. In the membrane pump the sludge enters a chamber through a ball valve in the bottom and leaves it through a similar valve in the top of this chamber. There is no plunger in the chamber, but its capacity is decreased and increased by the inward and outward pulsations of flexible membranes fastened over openings in the walls of the pressure chamber, which pulsations are produced by surges of water against the membranes, due to the reciprocating movement of a pump plunger in the mass of water. The operation is the same as that of a diaphragm pump except that the diaphragm is moved by water instead of a lever. At Hannover two such pumps were used in 1911 to lift sludge nearly 43 ft. into tanks above centrifugal drying machines. Each pump was rated at 5283 gal. per hour. The advantage claimed for the pump is that there are no finished metal surfaces in contact with the sludge and repairs are simple.

DISPOSAL AT SEA

For communities near the sea, carrying sludge in boats out to deep water and dumping it where no contamination of the adjacent shores is possible is often a satisfactory method of disposal. In Great Britain it is practised at London, Manchester, Southampton, Dublin, Glasgow and Salford, and in Germany at Hamburg. In the United States, Providence disposes of pressed sludge cake at sea, and the contents of the deposit sewers of the Boston, Mass., main drainage works are similarly disposed of.

The cost of such disposal in Great Britain and Ireland was investigated by the Royal Commission on Sewage Disposal, with the results given in Table 108. The following information regarding the methods at the cities mentioned in the table is mainly from the same source.

At London, a fleet of 6 steamers is engaged in transporting the sludge

from storage tanks at the treatment works at Barking and Crossness to a portion of the Thames estuary from 45 to 58 miles distant. The steamers are about 280 ft. long, 38 ft. wide and 14 ft. deep, and have a nominal capacity of 1000 long tons. The sludge has been settled and strained before it is stored. The steamers make 10½ trips a week, one being laid up for overhauling all the time. A steamer begins to discharge sludge when it reaches the disposal grounds, and runs out about 9½ miles while her load is being dumped. An investigation of the effect of this dumping was made by Prof. Frank Clowes, who drew the following conclusions from the results:

TABLE 108.—SUMMARY OF FIGURES RELATING TO COST OF SEA DISPOSAL
(From Fifth Report Royal Sewage Comn., page 167; adapted to American money and tons of 2000 lb.)

Place	Year or years to which figures refer	Ave. annual quantity sludge deposited at at sea, tons	Ave. percentage of water in sludge actually deposited	Total cost of sea disposal incl. cost of filling ship, int. and sinking fund, steamer dues and all other charges			Remarks
				Per ton sludge actually deposited.	Per ton of dry solid matter	Per ton of sludge containing 90 per cent. of water[1]	
Dublin....	1906–7	128,300	90.0 ap.	$0.09	$0.91	$0.09	No harbor dues and short distance to dumping area.
Glasgow..	1906–7	341,600	86.8	0.10	0.75	0.07	Large quantity of thick sludge. Loan repayable in 60 years.
London...	1903–6	2,838,100	92.0	0.08	1.02	0.10	Very large quantity, but thin sludge.
Manchester	1903–5,7	188,700	86.0 ap.	0.17	1.25	0.13	Heavy canal dues.
Salford...	1902–6	152,300	79.0	0.17	0.82	0.08	Heavy canal dues. Very thick sludge.
Southampton.	1906–7	95,600	90.0 ap.	0.30	3.04	0.30	By contract.

[1] These are figures obtained by calculation, on the assumption—which is probably not quite correct—that it is as easy to deal with dense as with liquid sludge. There would probably be more wear and tear of machinery in the case of dense sludge, but on the other hand the same amount of solid matter in the form of liquid sludge might mean more steamer capacity. The figures in this column must therefore be taken as giving only an approximate idea of comparative costs.

"A study of these tables shows that there are on an average over 100,000,000 bacteria per cubic centimeter in the sludge discharged by the Council's steamers in the Barrow Deep, and that about half this number are probably

of intestinal origin, since they are capable of growing at blood heat (37° C.); but that immediately after the discharge of the sludge the number in the water of the Barrow Deep is less than 2500 per cubic centimeter, and that the number of intestinal bacteria is generally less than 10 per cubic centimeter, indicating a reduction of about 99.9 per cent. in the total number of bacteria and of intestinal bacteria. Further, that the number of bacteria in the water which is carried out of the Barrow Deep toward the mouth of the Thames by an incoming flood tide, is still further reduced to an average of 440 per cubic centimeter, and that the number of intestinal bacteria has also further decreased. . . .. The results of these experiments appear to indicate that sea water[1] does not exert any marked inhibitory effect upon the life of bacteria." (Fourth Report, vol. ii, pages 108 and 110.)

At Glasgow, at the Dalmuir works, the sludge is stored in an elevated tank holding about 1500 long tons. As delivered to the tank it has about 93 per cent. moisture, but as sent to sea it contains about 86 per cent. of moisture. One steamer holding 1200 long tons is used. The sludge is discharged from the tank through two 16-in. pipes with swivel arms and flexible spouts into closed compartments in the vessel. The sludge is discharged about 40 miles from the works in 80 fathoms of water.

At Manchester, the sludge from the Davyhulme treatment works is taken on a steamer through the ship canal and out to sea, a total distance of about 61 miles, before it is dumped.

At Southampton the sludge is removed by contract in a 114-cu. yd. barge towed by a tug to a dumping place 25 miles distant.

At Salford, the sludge is pumped from 2 tanks through an 18-in. pipe into a steamer of 600 long tons capacity. It makes 5 trips a week to dumping grounds 60 miles distant. The tanks are 100 ft. in diameter and 9 ft. deep at the center, and together hold about 3000 long tons. Each of the sludge pumps has a rated capacity of 200 tons per hour. The steamer is 170 ft. long, 32 ft. wide and 11 ft. deep when loaded. The sludge is carried in 4 tanks and is discharged through eight 18-in. valves.

At Dublin the sludge is transported 10 miles in a steamer and deposited in about 90 ft. of water.

An entirely different form of sludge disposal in salt water is described by Dr. Gilbert J. Fowler in Appendix 5, Fifth Report of the Royal Commission on Sewage Disposal, page 555. At Shotley an institutional plant for 2000 persons had septic tanks and trickling filters, discharging their effluent over the foreshore. The sludge and some sewage, amounting to about 9600 gal., were discharged into the sea at suitable stages of the tide. The sludge valve was opened at or just after high tide and the course of the discharged sludge observed. As a

[1] This refers to comparative action of sea and fresh waters.

result of the investigation, Fowler concluded that if sludge consisting of finely comminuted solids, such as accumulate at the outlet end of a septic tank, is discharged at the beginning of ebb tide, under conditions like those at Shotley, while a temporary discoloration and pollution of the water takes place in the track of the sludge, yet at no time is the aeration reduced appreciably between tides, and finally the solids are so widely distributed that no visible effect is produced, although the actual transformation of the organic matter requires considerable time. Fowler was convinced that, so far as these investigations were concerned, nitrification did not proceed so quickly in salt as in fresh water, and oxidation of nitrogen reached only the stage of nitrite formation. More odor was caused by mixing sludge with salt than with fresh water, and disposal in sea water should be adopted cautiously on that account, for the addition of more than 0.5 per cent. of sludge was likely to cause a nuisance.

At Providence, R. I., the sludge is pressed before it is loaded on the sludge barge. The latter is 138 ft. long, 38 ft. deep and draws 3 ft. 3 in. empty and 9 ft. loaded. The freeboard when loaded is 2 ft. 6 in. There are 6 hoppers holding 148 cu. yd. each. The scow cost $18,191 and a specially constructed wharf for its use cost $18,607. For some years the scow was towed by a contractor in connection with his dredging scows. The distance out and back was 28 miles, and the contractor received $50 for the round trip. The U. S. inspector's fee was $1, and the scowman was paid $1.85 a day. The total per trip was $52.85, and as the average load was 892 cu. yd., the cost per cubic yard was 5.9 cents. In 1910, the total cost of disposal, including lime added to the sludge, pressing, supplies and repairs of pressing equipment, labor, light, heat, power and towing, was reported by City Engineer Clapp at $2.62 per ton of solids in the sludge, which had a moisture content of 72.4 per cent.

The advantage of carrying pressed sludge is that, under some local conditions, it need not be carried so far out to sea as wet sludge, and the volume of material to be transported is smaller. For handling dried sludge a hopper-bottomed barge, discharging its load by opening doors in the bottom, can be used. This type of barge has been well developed for dredging work.

SLUDGE PRESSING

In order to reduce the volume of sludge so as to make its disposal easier, part of the water in it is removed by filter presses at a number of works. The Royal Commission on Sewage Disposal stated in its fourth report that this was done in 21 British cities. It is practised in Potsdam, Spandau and Tegel in Germany and at Providence and

TABLE 109.—RESULTS OF SLUDGE PRESSING AT WORCESTER, MASS., 1899 TO 1912

Year	Sludge		Press cake		Solids		Pounds of lime added per 1000 gal. sludge pumped	Cost of operation			
	Thousand gallons pressed	Solids, per cent.	Cake, tons	Solids, per cent.	Total, tons	Tons per 1,000,000 gal. sewage treated		Total	Per 1,000,000 gal. sewage treated	Per ton solids	Per ton cake
1899......	52,600	2.99	33,101	26.1	6,606	1.13	11.1	$30,683.73	$4.92	$4.64	$0.93
1900......	39,487	4.42	27,286	28.0	7,299	1.98	21.2	24,933.31	6.76	3.42	0.91
1901......	24,920	5.25	20,152	27.0	5,450	1.74	20.0	18,477.59	5.89	3.39	0.92
1902......	20,905	8.01	22,059	31.6	6,975	1.52	30.3	23,808.05	5.20	3.41	1.06
1903......	24,527	7.44	25,088	30.3	7,608	1.45	33.5	25,766.71	4.91	3.39	1.03
1904......	24,331	6.93	24,332	28.9	7,026	1.66	28.5	24,640.18	5.83	3.51	1.02
1905......	15,419	9.80	19,448	32.2	6,297	1.71	53.5	23,344.13	6.33	3.71	1.20
1906......	15,761	8.09	16,959	31.3	5,310	1.25	52.2	20,171.91	4.74	3.80	1.19
1907......	17,979	7.22	18,240	29.6	5,407	1.19	41.5	22,392.19	4.92	4.14	1.23
1908......	12,074	8.20	12,987	31.8	4,126	1.00	49.4	15,819.91	3.85	3.83	1.22
1909......	12,606	6.93	11,500	31.7	3,642	0.99	44.7	15,760.34	4.28	4.33	1.37
1910......	12,244	8.20	12,867	31.6	4,182	1.17	53.5	16,208.41	4.53	3.88	1.26
1911......	12,981	7.52	13,469	30.2	4,066	1.13	53.5	16,608.27	4.60	4.08	1.23
1912[1]......	10,695	7.54	11,357	29.6	3,358	1.20	54.6	14,867.84	5.31	4.43	1.31

[1] *Note.*—On account of lack of funds, 5,900,000 gal. of sludge, containing 1701 tons of dry solids, were pumped onto the old sludge beds. In estimating the tons of solids per 1,000,000 gal. of sewage treated and the cost of operation per 1,000,000 gal. of sewage treated, the proportion of sludge pumped to the sludge beds was taken into consideration. During storms a large amount of sewage is discharged through overflows and never reaches the sewers. A portion of the strongest sewage has been run to the filter beds without preliminary chemical treatment. The sludge reported in the table is almost wholly that from chemical precipitation. The costs include the cost of hauling the sludge ¾ mile on a narrow gage electric railway and spreading it on waste ground.

Worcester in the United States. Worcester sludge-pressing statistics are given in Table 109. The process is one used extensively in a great variety of industries. In sewage treatment work it is rarely employed except where chemical precipitation is practised, and the sludge generally requires the addition of lime to enable the process to be used. At Providence and Worcester from 20 to 30 lb. of lime are added to 1000 gal. of the sludge from chemical precipitation, and as much as 100 lb. has been required to prepare the sludge from septic tanks for pressing by modifying any pasty and acid conditions.

The most extensive available results of sludge pressing are those of English plants as recorded in the Fifth Report of the Royal Commission on Sewage Disposal (page 168) from which Table 110 has been adapted. The Commission stated that pressed sludge was usually disposed of to farmers at 12 cts. per long ton in favorable districts, but in large towns it had to be given away or a small fee paid for its removal. The Commission reached the following conclusion:

"In the case of large towns, the cost of the disposal of sludge by pressing is about the same as the cost of depositing it at sea. For small towns or for places where septic tank sludge has to be disposed of, the cost is considerably greater. In all cases the cost turns very much upon whether there is a ready sale for the pressed cake, and this is dependent upon having facilities for carting or sending it by rail to outlying districts. The pressed cake is worth much more than 6d. (12 cts.) per ton, judged by its manurial constituents; the reason that it fetches such a low price is the relatively high cost of carriage upon a mixture containing of necessity a large proportion of water, grit, and carbonaceous matter."

At Worcester the sludge cake is dumped in a depression at some distance from the works and has not proved offensive. Some has been used by neighboring farmers, who have hauled it distances of about 4 miles in some cases; in some years over a thousand two-horse loads have been taken away. The pressed cake produced at Providence is dumped at sea, as already explained.

Filter Pressing.[1]—The press used in this process consists of 2 heavy castings, known respectively as the head and tail ends, connected by parallel side rods which carry the plates and the follower. To the head end is attached the piping through which the sludge is forced into the press, while the tail end carries the apparatus used in opening and closing, as shown in Fig. 117.

The plates are of cast iron, generally about 36 in. in diameter by 3 in. thick for sludge pressing, with recessed and corrugated faces, and have an opening 6 in. in diameter in their center. The follower is of the same shape as the plates, but is made much thicker to withstand

[1] For this description the authors are indebted to Julius W. Bugbee, superintendent and chemist of the Providence sewage treatment works.

TABLE 110.—COST OF PRESSING SLUDGE

(Unless otherwise stated, cost of pressing is taken as beginning when sludge is out of sedimentation tanks and ready to be pumped into presses. Cost of pressing taken as ending when pressed cake is in trucks below presses. Compiled from Fifth Report of the Royal Commission on Sewage Disposal, page 168.)

Place	Year	Population	Kind of sludge pressed	Lime added to sludge for pressing, in per cent. of		Cost of pressing sludge, excl. int., etc.,[1] per ton (2000 lb.)		Cost of pressing sludge, incl. int., etc.,[2] per ton (2000 lb.)		Remarks
				Wet sludge, per cent.	Pressed cake, per cent.	Wet sludge	Pressed cake	Wet sludge	Pressed cake	
Hanley	1907	66,000	Septic detritus	0.35	1.43	$0.08	$0.34	$0.10	$0.41	Dumped at works. Milk of lime added hot, as sludge is pumped into well. Mixture then stands several days before pressing. Septic tanks cleaned monthly.
Glasgow (Dalmarnock)	1906–1907	300,000	Chem. precip.	0.416	2.50	0.07	0.42	0.08	0.50	Pressed cake sold to farmers at 14 cts. to 21 cts. per ton bulk. Farmer pays hauling.
Leyton	1906–1907	120,000	Chem. precip.	0.75	3.75	0.09	0.43	......	......	Most of cake burnt in destructor; rest given to farmers. 1907 entered into contract; contractor to pay 11 cts. ton for cakes loaded on truck on railroad.
Stockport	1906–1907	91,500	Chem. precip.	0.85	3.25	0.11	0.46	0.14	0.54	Given away to farmers.
Wolverhampton	1906–1907	101,000	Chem. precip.	0.49	2.10	0.10	0.43	0.12	0.50	56 per cent. by dumping; rest barged to farmers. Farmers pay $4.86 per boat load. Average loss to corporation = 25 cts. per boat load of 2 long tons = 11.1 cts. per ton of cake.
Bolton	1906	170,000	Chem. precip.	1.12	5.6	0.10	0.27	0.18	0.91	Removed by farmers, corporation paying 4 cts. per ton pressed cake.
Bury	1906–1907	50,000	Chem. precip.	0.67	3.62	0.10	0.50	......	......	Given away to farmers. Assistance given in loading.[3]
Richmond (Surrey)	1907	62,885	Chem. precip. and septic	ap. 1.0	ap. 5.0	0.10	0.50	0.13	0.64	Cake dumped into barges owned by contractor, who takes them away. Sold by him to farmers in Essex and Kent. Cost[4] to Richmond Main Sew. Bd., 1907, 33 cts. per ton pressed cake, 7 cts. per ton wet sludge.

[1] Cost of pressing sludge, including labor, fuel, lime, press-cloths, repairs and renewals, but excluding interest and sinking fund on pressing plant.

[2] Cost of pressing sludge, including labor, fuel, lime, press-cloths, repairs and renewals, and also interest and sinking fund on pressing plant.

[3] Figures of cost for Bury include cost of sludging precipitation tanks. Very small, however, involving labor of but 1 man.

[4] It should be noted, with regard to capital expenditure, that engine plant and machinery is in duplicate. Plant is only worked 1 shift of 10 hours per day; if it were worked in 2 shifts, cost of pressing, including interest and sinking fund, would be 12 cts. per ton wet sludge, or 36 cts. per ton pressed cake.

TABLE 110.—COST OF PRESSING SLUDGE.—(*Continued*)

(Compiled from Fifth Report of the Royal Commission on Sewage Disposal, page 168)

Place	Year	Population	Kind of sludge pressed	Propor. lime added to sludge for pressing, in per cent. of		Cost of pressing sludge, excl. int. etc.,[1] per ton (2000 lb.)		Cost of pressing sludge, incl. int., etc.,[2] per ton (2000 lb.)		Remarks
				Wet sludge, per cent.	Pressed cake, per cent.	Wet sludge	Pressed cake	Wet sludge	Pressed cake	
Ealing (Southern Wks.)	1906–1907	40,000	Chemical precipitation and settling tank.	5.0	20.0	$0.19	$0.93	$0.23	$1.04	Cake dropped into trucks and taken by lift to destructors. Sludge pressed especially for burning.
Maidenhead.......	1906–1907	14,000	Chemical precipitation	......	4.2	......	0.93	......	......	Pressed cake and some air-dried sludge sold to farmers and market gardeners at 11 cts. per cart-load.
Nelson............	1906–1907	37,000	Septic and settling tank	......	9.6	......	0.98	......	1.29	Given away to farmers. Labor cost in connection with sludge pressing is estimated.

[1] Cost of pressing sludge, including labor, fuel, lime, press-cloths, repairs and renewals, but excluding interest and sinking fund on pressing plant.
[2] Cost of pressing sludge, including labor, fuel, lime, press-cloths, repairs and renewals, and also interest and sinking fund on pressing plant.

TABLE 110.—COST OF PRESSING SLUDGE.—(*Continued*)

(Compiled from Fifth Report Royal Commission on Sewage Disposal, page 168)

Place	Year	Population	Kind of sludge pressed	Lime added to sludge for pressing, in per cent. of		Cost of pressing sludge, excl. int., etc.,[1] per ton (2000 lb.)		Cost of pressing sludge, incl. int., etc.,[2] per ton (2000 lb.)		Remarks
				Wet sludge, per cent.	Pressed cake, per cent.	Wet sludge	Pressed cake	Wet sludge	Pressed cake	
Hyde	1903	32,000	Chem. precip.	1.25	3.75	$0.18	$0.54	$0.27	$0.82	Removed by farmers free, except surplus for which 14 cts. ton paid to clear it away. Cost includes tank sludging. Small item, however.
Great Harwood	1906–1907	22,500	Chem. precip.	......	3.0	......	0.54	......	0.77	Sold to farmers at 10 cts. per ton.
Chorley	1906–1907	26,500	Chem. precip.	0.42	4.25	0.06	0.57	......	......	By sale to farmers; 36 cts. per 1-horse load = about 3360 lb., 42 cts. per 2-horse load = about 4480 lb.
Wimbledon	1906–1907	52,500	Chem. precip.	1.0	5.0	0.14	0.66	0.16	0.78	Burying and burning.
Royton	1906–1907	12,500	Chem. precip.	......	4.14	......	0.68	......	0.88	Given away to farmers, who remove it. Cost includes sludging tanks. Steam for pressing raised by destructor using but little coal.
Leigh and Atherton	1906–1907	63,600	Chem. precip.	......	4.3	......	0.73	......	0.88	Part placed on portions of sewage farm not used for sewage, part sold to farmers at 21 cts. per 1-horse load and 32 cts. per 2-horse load. About 10 cts. per ton on average. Cost of pressing, including interest and sinking fund, is estimated.
Middleton	1906–1907	25,600	Settling tank	0.67	4.6	0.43	0.84	0.15	0.98	Given away to farmers.
Willesden	1905–1907	66,000	Chem. precip.	2.2	13.35	0.14	0.80	0.15	0.87	Mostly dumped at sewage farm. In 1907 some pressed cake given farmers, who paid freight. Large percentage of lime due to laundry waste and grease.
Burnley	1905–1906	80,000	Septic and detritus	3.4	15.5	0.23	0.93	0.25	1.00	Sold to farmers for 16 cts. per ton.
Colchester	1906–1907	41,000	Chem. precip.	2.8	11.2	0.23	0.93	0.28	1.09	Pressed cake sold farmers, at 21 cts. per cart-load.

[1] Cost of pressing sludge, including labor, fuel, lime, press-cloths, repairs and renewals, but excluding interest and sinking fund on pressing plant.
[2] Cost of pressing sludge, including labor, fuel, lime, press-cloths, repairs and renewals, and also interest and sinking fund on pressing plant.

the pressure of the closing apparatus. When the plates have been covered with canvas, termed "clothing," they are forced tightly together and against the head end by pressure applied to the follower, thus forming a series of canvas-lined chambers, each connected with the next one by a 6-in. circular opening. Sludge is then forced into these

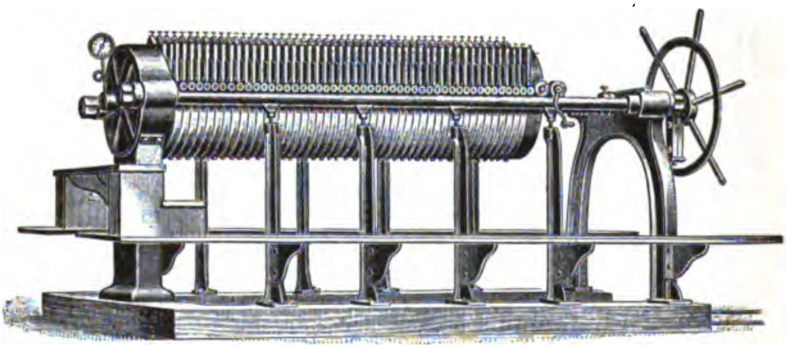


FIG. 117.—Filter press for sewage sludge (Bushnell).

chambers under pressure. The water passes through the cloth, follows the corrugations on the face of the plates, and finally passes through a drainage duct located at the bottom or one of the lower corners of the plate, while the solids are retained by the cloth.

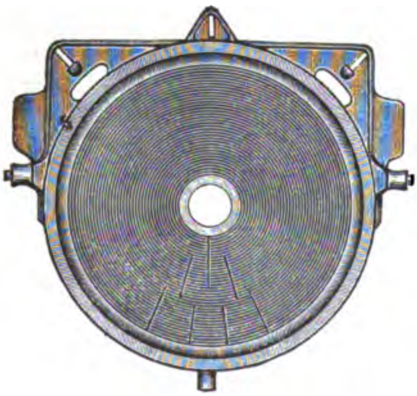


FIG. 118.—Plate used in filter press (Bushnell).

The main piping should be not less than 8 in. in diameter, with long turns and flanged joints.

The head-end inlet pipe should be 6 in. in diameter.

The inlet valve should be an angle valve of the plunger type.

The core valve should be a 6-in. gate valve, connected to a tee placed between the inlet valve and the head-end casting, for the purpose of draining off any liquid sludge in the press, just before opening. This is necessary only with very large units.

The side rods should be rolled steel, 1½ × 9 in., planed on the upper edge and fitted into recesses in the head and tail ends, where they are bolted in place. Round rods are also used. There should be from 50 to 75 plates per press. They should be 36 in. square, 3 in. thick, and

recessed about 3/8 to 1/2 in. on each face. Some plates have pyramidal corrugations, formed by 2 sets of grooves, running horizontally and vertically across the plate. This forms a series of pyramids on whose points the filter cloth rests, the grooves forming channels to carry the filtrate to the drainage duct. Other types of plates have circular corrugations, as shown in Fig. 118.

The drainage duct is a horizontal hole 1/2 in. in diameter and 6 in. deep, drilled or cored in the bottom or one of the lower corners of the plate, parallel to, and midway between, the filtering surfaces, and connecting with them by means of a slot or slots, about 1/8 × 4 in., in each face of the plate. This duct must be straight, as it should be frequently cleaned to remove the lime which is continuously deposited from the filtrate. Lugs cast on each side of the plate serve to suspend the plate on the side rods, and also form handles for use in pulling the plates when cleaning the press.

The newer forms of plates are suspended directly on these lugs, instead of being carried on rollers, as in the older types.

The closing apparatus is attached to the tail-end casting and serves to force the plates and follower firmly against the head end, and to hold them in position during the operation of filling the press. The apparatus may be a hydraulic ram, or a hand-operated or motor-driven screw.

Clothing for a 36-in. press is made from cloth 40 in. wide by cutting strips a little more than twice the vertical length of the plate, and in each of these strips cutting 2 holes 6 in. in diameter to correspond with the central hole of the plate. The cloth is then hung over the top edge of the plate and a clamp is put through the holes in the cloth and the opening in the plate, and drawn tightly together, thus completely covering both surfaces of the plate with canvas.

The material generally used is either United States standard army duck, hose duck or chain duck. Comparative tests made at the Providence plant resulted in the selection of United States standard army duck, 11 oz. per yard, 40 in. wide, as the most economical material. Clothing made from this material will last from 6 to 9 weeks, the presses being in operation 10 hours per day, and each press being cleaned 12 times in each 10-hour period. Some abrasion of the cloth occurs by the striking of one plate against another during cleaning, but aside from the wear due to this cause, the cloth appears to deteriorate nearly as rapidly when used a few hours each day as when it is in continuous service.

The causes which contribute to this are fermentation of press liquor retained in the cloth, burning by partially slaked particles of lime, and rust from the plate surface. In some pressing plants, the cloths are removed from the presses at regular intervals, washed and

repaired, and returned to the presses, while the practice in other places is to keep the cloth in place until it is unfit for further use. The washing is done in a barrel washer, using slightly warm water and no soap, to avoid shrinkage, and the cloths are then dried in a centrifugal basket.

If very long presses are used, the sludge is made to drop into a trough extending below the entire length of the press, and from this trough it is scraped into cars by a conveyor, but if presses of less than 75 plates are employed, cars can be placed directly beneath the press, the sludge falling into them through a hopper.

The ordinary type of V-bottom dump cars is the most convenient for use in hauling pressed sludge, as the cake slides from them very rapidly, and the construction of the car prevents leakage of wet sludge during transportation.

CENTRIFUGAL SLUDGE MACHINES

Many attempts were made to employ centrifugal machines to dry sludge before one was developed which was practicable. The difficulty was due in part to the high cost of drying where the machine was stopped, after each charge had been treated, in order to remove the dry material and admit fresh, and in part to the peculiar behavior of sludge in a centrifuge. The solid particles are carried outward and formed into a tough, hard mass which is not easily scraped from the walls of a drum of a centrifugal separator.

The first practicable sludge machine was developed by City Engineer Schaefer of Frankfort and Director ter Meer of the Hannover Machine Co., of Hannover-Linden. It is shown in Fig. 119. The sludge is admitted to the top from an elevated tank and passes down through a hollow shaft to the runner of the centrifuge. This runner has 6 radial cells, 1 radial side of each cell being formed of a sieve, so as to permit the water in the sludge to escape while it is being whirled. The sludge is admitted to these cells from the hollow central shaft through openings which may be closed by gates, and it is thrown out of the cells through openings in the outer ends of the cells, which can be opened or closed by similar gates. These outer gates are shut and the inner set open while the cells are filling with sludge. When they are full the inner gates are closed and the machine kept whirling until the operator considers the sludge is in proper condition for discharge. The outer gates are then opened and the centrifugal force of the revolving runner throws out the sludge against the walls of the chamber. The impact breaks it up and it drops through a hopper to a belt conveyor by which it is removed. The water which is driven off from each cell is collected by a plate behind each sieve and guided to a down-drain, through which it drops into a trough. The gates are moved by pressure-oil and it is

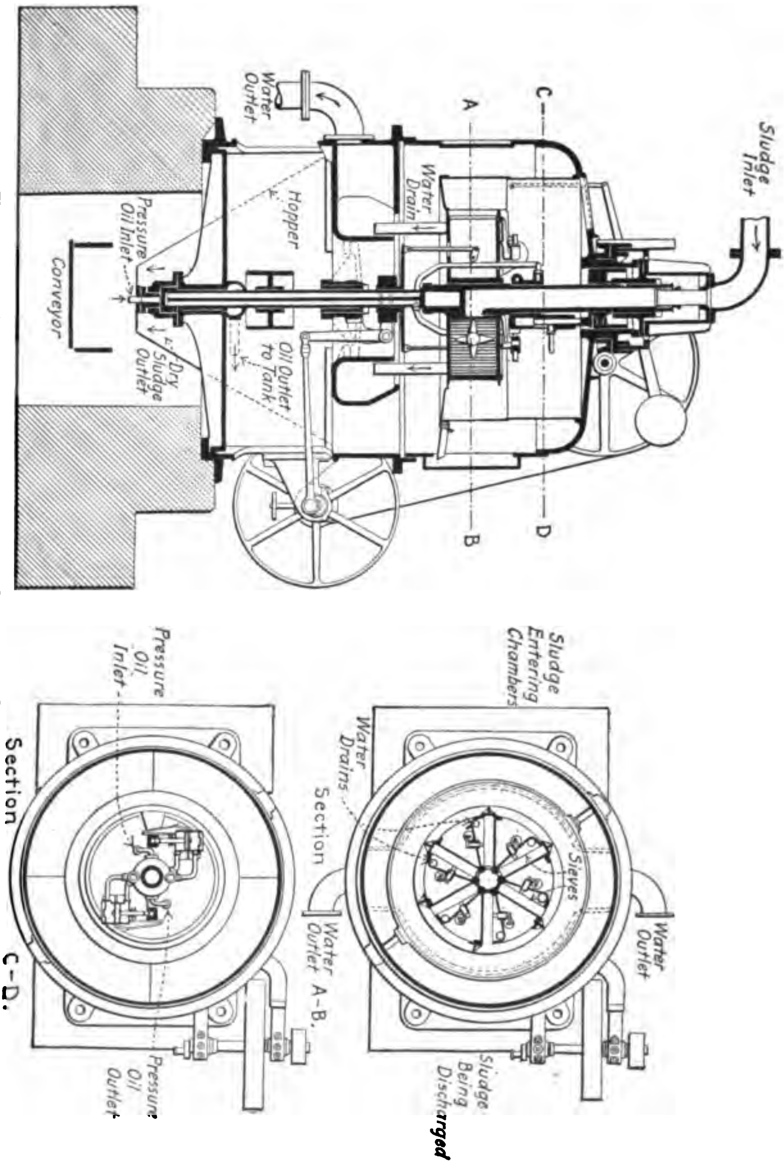


Fig. 119.—Centrifugal machine for drying sludge (Schaefer—ter Meer).

unnecessary to check the speed of the runner in order to charge and unload the cells.

The first two of these machines were installed at Harburg in 1907. The sewage is screened through a rack with 5/16-in. openings and then passed into vertical sedimentation tanks. In 1909, from 530 to 700 cu. ft. of sludge with 88 to 95 per cent. of water was treated daily. This was dried in 7 to 8 hours in 1 centrifuge. In a test of the plant by Reichle and Thiesing (Mit. Kgl. Prüfungs-Anstalt, vol. x) it took about 2½ minutes at a speed of 750 r.p.m. to dry the sludge. The dried material contained considerably less grease than the raw sludge. The percentage of moisture in the raw sludge tested was about 92 while that in the dried sludge ranged from 69.7 to 74.2. Of the organic matter in the raw sludge about three-fifths remained in the dried sludge, and the remainder was carried off by the water, which had a foul odor.

The sewage of Hannover, where the second installation of centrifugals was made, is screened, pumped and then allowed to settle in tanks. These basins number 12, 4 being in reserve. Each is 148 ft. long, 26 ft. wide, 7.2 ft. deep at the outlet end and 9.8 ft. at the other, except at the end wall where there is a transverse pit 12 ft. deep. The sludge is pumped to 2 raised tanks, each containing a stirrer, from which it flows to 4 centrifugal machines, which cost about $21,250. The fresh sludge contains about 92 per cent. of water and the dried sludge from 55 to 60 per cent. It is necessary to clean the sieves of the machines daily. This plant was started in 1908 and was in excellent condition when inspected by one of the authors in 1911.

SLUDGE DRYING BEDS

Most sludge is discharged from the tanks where it settles to beds where it dries in the open air. Part of the water sinks into the soil and part evaporates; if the soil is not open and the sludge is greasy and dense, the liquid condition may remain for some weeks. The beds are preferably porous and well underdrained at a depth 24 to 30 in. with 4 to 6-in. open-joint tile as in intermittent filters (Chapter XVI). The effluent from the beds must be kept under observation. It may be necessary to return it to the works for treatment with the raw sewage if of a questionable nature and there is no opportunity for filtering or otherwise treating it independently of the sewage.

In Massachusetts, where intermittent filtration is much in vogue, sludge is disposed of on beds similar to the filter beds, and sometimes even on one of the latter, reserved for the purpose. For example, at Marlboro, the sludge is dried upon sand beds having an area of 3 sq. ft. per person served by the sewer system. At Gloversville, N. Y., where large quantities of sludge were expected because of the industrial

wastes, an area of about 6.5 sq. ft. per person was provided. Table 111 gives the areas of the beds in a number of English and European cities.

At Plainfield, N. J., the sludge from septic tanks was run onto a 2-ft. bed of sand having an effective size of 0.2 to 0.25 mm., underdrained with 3-in. tile. There was considerable odor while the sludge was flowing, but after 2 or 3 days none was noticeable except in the immediate vicinity of the bed. The sludge required 3 to 4 months to dry, and when dry most of it was removed by farmers, the remainder being used on the city farm.

TABLE 111.—SIZE OF SLUDGE DRYING-BEDS

(From "Sewage Sludge," Elsner, Spillner and Allen, page 63)

Place	Total area, acres	Sq. yd. of area		Sq. ft. per capita	Method of treatment
		Per cu. yd. sludge per day	Per capita		
Brieg	0.74	275	0.14	1.3	Sedimentation wells.
Langensalza	2.47	458	1.00	9.0	Sedimentation wells.
Stargard i.P.	6.17	1,905	1.11	10.0	Sedimentation wells. Much land in reserve.
Ohrdruf	0.16	....	1.2	10.8	Sedimentation wells.
Elberfeld	6.00	....	0.10	0.9	Sedimentation tanks.
Frankfort	12.35	183	0.17	1.5	Sedimentation tanks.
Cassel	2.47	137	0.08	0.7	Sedimentation tanks.
Munich-Gladbach	6.79	420	1.10	9.9	Sedimentation tanks.
Halberstadt	1.40	136	0.17	1.5	Septic tanks.
Mülheim a. R.	0.49	131	0.06	0.5	Septic tanks.
Unna	0.12	229	0.06	0.5	Septic tanks.
Imhoff tanks	....	46–55	0.02	0.2	Septic action.
Rochdale	0.22	....	0.18	1.6	Sedimentation and septic tanks.
Leeds-Knostrop	22.23	275	0.25	2.3	Precipitation with lime.
Accrington	2.22	916	0.22	2.0	Septic tanks.

The sludge from a storage reservoir at Brockton, Mass., was pumped for a number of years to sand beds. After drying it was at first raked into piles and burned over wood fires, which caused many protests. In 1898 it was sold to farmers and when the quantity became large a few years later, it was given away to encourage them to remove it. In

1908, a 5-year contract was made with a man living near the beds, to let him have the sludge without charge, in consideration of its immediate removal when raked up.

TABLE 112.—SHOWING RESULTS OF AIR-DRYING SLUDGE IN EARTH LAGOONS
(From Report of the Sewage Testing Station, Philadelphia, 1911, page 178)

Source of sludge	Time, days	Depth, inches	Cu. yd. lagoon	Moisture in sludge, per cent.	Rain-fall, inches	Sludge per acre, cu. yd.
Sedimentation tank 12; influent, screened sewage.	0 26 49	12.20 7.67 3.50	3.60 2.50 1.04	82.8 57.0 51.6	0 0 0.43	1600 1000 470
Sedimentation tank 12; influent, screened sewage.	0 62	13.50 7.00	4.00 2.10	90.1 61.0	0 3.14	1800 950
Contents of the sludge digestion tank in first experiment.	0 23 44	12.00 2.67 2.67	3.50 1.80 1.80	96.5 60.4 51.6	0 0.43 0.82	1600 360 360
Sedimentation tank 12; influent, crude sewage.	0 59	12.00 4.70	3.50 1.40	88.7 62.8	0 2.59	1600 640

Imhoff Tank Sludge.—The sludge from Imhoff tanks in satisfactory working condition is dried very easily, partly because of its low amount of water and partly because of the large quantity of entrained gases in it. Good sludge of this kind is black, fibrous and not particularly slimy, and has a tarry odor, while poor sludge is slimy and has the odor of septic sludge. The bubbles of gas expand when the material is run onto the beds, thus facilitating the drying-out process. Fig. 120 shows the sludge from one of the larger tanks in the Emscher District being run onto a sludge bed, Fig. 121 shows a bed of dried sludge, and Fig. 122 gives a very good idea of the facility with which the dried sludge is handled. The sludge channels in this district have a slope of only about 1:40. The following description of the behavior of Imhoff tank sludge in drying is from Gault's report (1912) on Fales' experiments with an Imhoff tank at Worcester:

"When first drawn onto the drying bed, the Imhoff tank sludge became completely covered with gas bubbles, due to the release of pressure on drawing from the tank. The escaping gas lifted the solids, leaving a comparatively thin liquid at the bottom, just as in the case of (experiments with)

the glass cylinders, where, inside of 24 hours, from 15 to 30 per cent. of clear liquid remained at the bottom. This clear liquid filtered away within a day or two, reducing the volume of the sludge correspondingly. Under

Fig. 120.—Sludge running onto sludge bed, Emscher district.

favorable weather conditions, the surface of the sludge on the second day presented a thin unbroken crust. When the sludge was stirred up, however, much gas escaped, making a sound like a man puffing on his pipe, and within

Fig. 121.—Dried sludge on sludge beds, Emscher district.

a few minutes the surface where it was broken looked as if it were strewn with blackberries, due to the gas bubbles in the sludge. The entire mass where disturbed, except for color, now resembled soft dough in which yeast is

very active. Within a few hours after this condition, cracks began to appear on the surface and gas escaped through these cracks, causing the sludge to become a seething mass. The evolution of gas soon subsided, although the sludge retained much gas for several days, rendering the sludge porous and spongy. The cracks gradually increased in number and deepened, thereby facilitating the drying. The bottom portion was the last to dry, the time required to dry out depending upon the depth of sludge" (page 34).

In connection with the experiments at Worcester, comparative tests were made of the behavior during drying of Imhoff-tank, chemical precipitation and plain sedimentation sludges. The last named was very coarse and contained a large amount of hair from a local tannery. It formed a mat over the surface, which did not crack readily, so that the air failed to penetrate it thoroughly. For this reason it did not dry

Fig. 122.—Removing sludge from bed, Emscher district.

so completely as the Imhoff-tank sludge, except when exposed in very thin layers, and rain falling while the sludge was drying was likely to undo the effect of 2 or 3 weeks' exposure. The chemical-precipitation sludge formed a multitude of cracks which gave it an advantage over the Imhoff-tank sludge when applied in a layer not over 6 in. thick. It did not lose its liquid condition so quickly as the Imhoff-tank sludge, but the cracks caused it to reach a condition suitable for removal quite as soon.

In the Emscher District the drying period, before the sludge is spadeable, is on the average 6 days, although under very favorable conditions the period may be as short as 2 or 3 days. On the other hand, but little can be done in the winter toward sludge removal, as was

indicated at Worcester, Mass. At this place, under favorable conditions, the sludge could be removed from the beds in 10 to 14 days. In one case in June, when but 6 in. was placed in the bed, it could be forked in 1 week.

Weather conditions are of great importance. If rain falls within 24 hours after running sludge on a bed, it tends to liberate the gas, making the sludge more compact and less porous, so that it does not dry out as rapidly as otherwise. If rain falls while the surface of the sludge is cracking, the water is absorbed as by a sponge, but if the sludge is nearly dry so that the cracks extend through the mass, the water soon drains away and the sludge dries again.

FIG. 123.—A sludge dump in the Emscher district.

One of the dumping grounds of the Emscher District is shown in Fig. 123. It will be seen that the track is laid on top of the sludge previously deposited. After a few months a considerable growth of vegetation starts, as may be seen by the illustration.

The area of sludge beds allowed by Imhoff is calculated on the basis of from 3 to 4 persons per square foot. These beds are constructed of 10 in. of slag and covered before each filling with a small layer of sand or grit taken from the grit chambers. From 6 to 12 in. in depth of sludge is run on at one time. Fig. 124 shows one of the German drying beds for a population of 100,000 persons. In the design of the Fitchburg, Mass., treatment works the size of the beds was based on the Imhoff rule, 1 sq. ft. for 3 persons. There an area of

about 0.4 acre was divided into 11 beds, each about 15 ft. wide and 111 ft. long, by 12 × 2-in. planks. The natural sand area has not been underdrained, but this work will be done if experience with the beds shows it to be necessary. A narrow-gage track with its ties flush with the surface of the sand runs down the center of each bed as shown in Fig. 125.

The Emscher District average practice may not prove entirely suited for average American conditions, owing to differences in the character of the sludge and in climatic conditions. Experiments at Worcester showed that about 0.021 cu. ft. of sludge per capita was produced daily. If the sludge is removed once a month, about 0.63 cu. ft. capacity, or at least 1 sq. ft. of bed per capita must be supplied, or about three times the area employed at Fitchburg. It should be observed, however, that the computation is based on a purely arbitrary assumption as to the necessary period for drying. Under some conditions, it might be that a much smaller area will suffice, and also that sludge can be removed from beds without waiting for it to dry thoroughly, although such removal will entail greater expense.

According to Spillner and Blunk ("Sewage Sludge," 1912, page 177) the fresh sludge that settles in the sedimentation chamber of an Imhoff tank in the Emscher District will have about 95 per cent. of moisture. After remaining in the sludge chamber for several weeks it will have an average of only 75 per cent. water and about two-thirds of the original amount of organic matter, the shrinkage in volume being about 84 per cent. This rotted sludge will lose about 40 per cent. of its volume while drying on the sludge bed, so that from 100 cu. yd. of fresh sludge only 9 cu. yd. of dried sludge remains.

In the experiments at Worcester, Mass., it proved practicable to remove the sludge while it contained as much as 70 per cent. of water, although the volume and weight of such moist sludge can be greatly reduced by further drying. The sludge removed weighed from 60 lb. per cubic foot when it contained 68 per cent. of water to 31¼ lb. with a water content of 8 per cent. The latter grade was produced by drying a thin layer of sludge for 4 weeks during very favorable weather. The average loss in volume at Worcester was 70 to 80 per cent. and the average loss in weight was 75 to 85 per cent.

Sludge from Secondary Sedimentation Tanks.—In many cases it is considered desirable to run the effluent from trickling filters into secondary sedimentation tanks, in order to remove the humus-like sludge it contains. This sludge at Worcester was dark brown, of the consistency of thick cream, and with a disagreeable fishy odor attributed to the multitude of angleworms in it. The humus sludge from the tank of the experimental trickling filter at the Chicago Stock Yards is described by Wisner and Pearse (Report on Industrial Wastes, page 183)

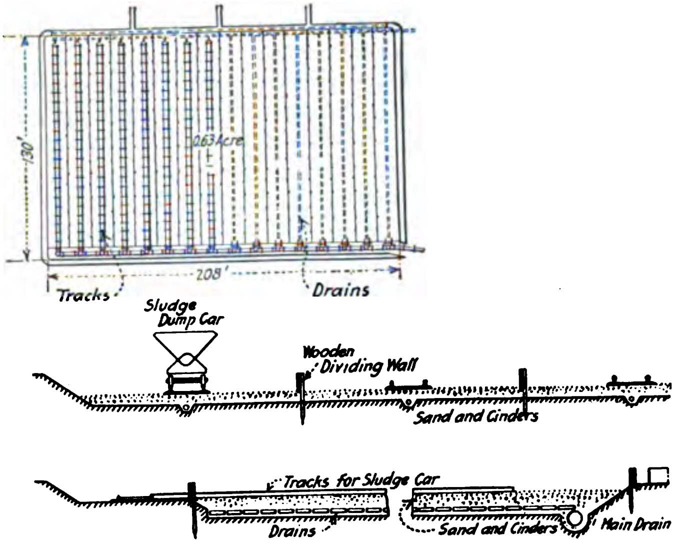


FIG. 124.—Sludge-drying beds for 100,000 persons (Imhoff).

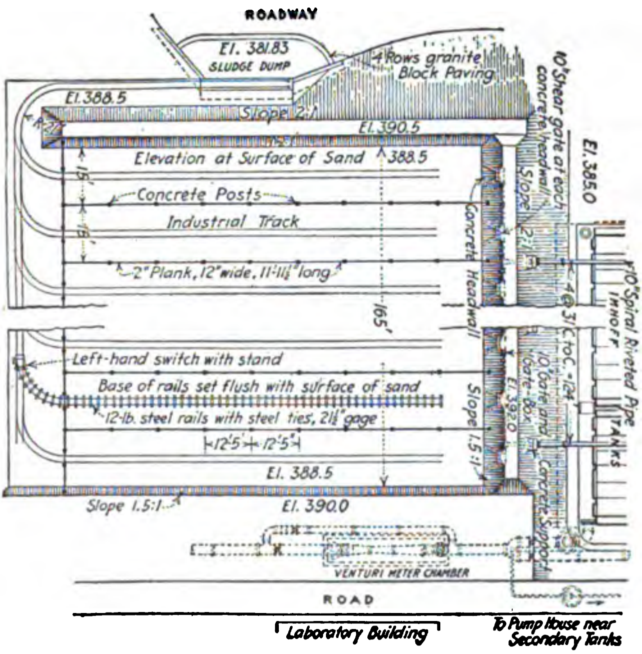


FIG. 125.—Sludge beds, Fitchburg, Mass.

as uniformly of a dark brown color, smooth consistency and odor of decaying vegetables.

The humus sludge from the Worcester experimental plant was turned on to a drying bed of sand and very soon acquired a light brown color on the surface. It dried very slowly on account of its gelatinous nature. When the weather was favorable, cracks appeared on the surface in a few days and the top sludge became a stiff jelly. The cracks increased in number and with their increase the drying extended deeper, until the cracks reached the sand bed. The sludge thereupon shrank greatly and was baked by the sun into hard brittle lumps.

In view of the disagreeable odor of this humus sludge at some plants, at the Fitchburg works provision was made to pump the contents of the secondary sedimentation basins into the Imhoff tanks, where the humus sludge is mixed with the Imhoff-tank sludge. At Baltimore the sludge from the secondary sedimentation basins was estimated by Frank (*Engineering Record*, July 4, 1914) to amount to about 0.45 cu. ft. every 6 months per inhabitant contributing. Like the sludge from the sedimentation tanks, it is pumped into sludge-digesting tanks, Baltimore being one of the few American cities where sludge treatment in special basins is practised. A new group of tanks for this purpose was planned in 1914, each tank being 38 ft. in diameter, 15 ft. deep to the bottom of the cylindrical upper portion, and provided with a conical bottom 9.5 ft. deep. The digested sludge will be dried on sludge beds designed on the basis of about 1 sq. ft. for about 10 inhabitants. The plans call for beds with 4 in. of sand on 11 in. of gravel or broken stone. Frank stated that the operation of the original secondary tanks has been similar to that of the preliminary sludge tanks receiving the sludge from the preliminary sedimentation basins. The latter are kept in operation until one of them shows signs of undesirable septic action, when the supernatant sewage is pumped off and the sludge pumped into the preliminary sludge tanks, concerning which Frank's comment was as follows:

"The operation of the present 3 sludge tanks has not, as yet, proceeded to a stage where it is possible to state with finality the degree of satisfaction they will eventually afford. They are but 13 ft. deep, rather shallow where compared with the depth of 25 to 30 ft. below water line of the usual type of Imhoff tank. The sludge in them possesses the normal characteristic odor of properly digested sludge, faintly like hot sealing wax, but often shows a rather high moisture content and dries somewhat slowly. It is hoped that prolonged operation of these digestion tanks will cause the decomposition process to acquire a higher moisture-reducing efficiency and the resulting sludge to have a shorter drying period. On the whole, the results thus far obtained with the separate sludge digestion tanks at Baltimore are decidedly gratifying. The writer knows of no plants, except such as use Imhoff tanks, where the results are better."

Travis Tank Sludge.—The sludge from the Travis tanks at Norwich is of a mucilaginous character, and could not be made into cakes in a filter press. The sludge was run for some time into trenches 18 to 24 in. deep and buried; one of the authors saw a trench opened after being covered for 4 to 6 months and considered the odor very offensive. The outer part was black and rather dry but the inside was pasty and had the original character and odor. The sewage contains at times a large amount of starch and brewery wastes, and is grayish white in color, and at other times is black. The water-supply of the town is hard and the sewage has a large amount of grease.

As a result of the unsatisfactory conditions attending this method of disposing of the sludge, the city made a contract with the Norwich Natural Manure Co. to take over the disposal for about $3000 a year, the cost to the city for burial having been about $2000 to $2500. This company uses the Eckenburg wet carbonizing process. This gives the sludge a somewhat granular nature, and in this form it parts with much of its water while standing in a tank. The settled sludge is run into filter presses and the cake dried in a rotary drier until it contains not over 10 per cent. of water. The dried cake is treated with benzine, which is drawn off later, partly as a vapor and partly as a liquid containing grease, dirt and water. The degreased sludge is pulverized and finds a ready market at about $12 per ton of 2240 lb. The benzine is recovered and the grease is treated with acid and sold at $35 to $50 per long ton.

LAGOONS AND TRENCHES

The method of disposal in lagoons consists in flooding an area to a considerable depth with sludge, which then slowly dries under the influence of digestion and evaporation.

At Reading, Pa., these lagoons cover about ¼ acre of land and are formed by throwing up dikes about 5 ft. high. Sludge is run into them to a depth of from 1 to 5 ft. After each cleaning, the contents of the lagoons shrink largely, due to draining, evaporation and septic action, and if the sludge is several feet thick a thick scum forms, covering a liquid sludge (*Engineering Record*, Aug. 13, 1910). At the Philadelphia Experiment Station studies with earth lagoons gave the results shown in Table 112, page 500. The English experience was summed up by the Royal Commission on Sewage Disposal in its fifth report as follows:

"There is, in our opinion, very little to be said for the lagooning of sludge. Once in the lagoon the sludge may take from 2 to 6 months to dry, according to the weather, and during the greater part of this time smell may be said to arise from it. Further, when the sludge is dry enough to be removed, it has still to be disposed of, and being usually moist and closely bound together it is not readily taken by farmers. The time re-

quired by the sludge to dry depends to a great extent upon the depth and the draining of the lagoon" (page 174).

In Birmingham, England (*Proc. Inst. C. E.*, vol. clxxxi), sludge is disposed of by running it into trenches about 3 ft. wide and 18 in. deep, which are then covered with earth to a considerable depth. After the sludge has dried it is worked into the soil by a steam plow. In order to reduce the amount of water to be handled, the material is pumped from the tanks to sludge-settling tanks, where the water content is reduced to about 80 per cent. and the supernatant liquid is again passed through trickling filters.

This method is also followed at Guildford and at the Withington works of Manchester, England. Watson thinks that ¼ acre of land is amply sufficient to deal with 1000 long tons (about 256,000 gal.) of wet sludge. (Fifth Report, Royal Commission on Sewage Disposal, page 172.) Climatic conditions and methods of working have considerable influence on the results. His experience has been that it is advisable to cut the trenches a month or two in advance of their being required, in order to dry the land as much as possible. From the experience at Guildford, C. G. Mason estimated that the following areas of land are advisable per 1000 long tons of wet sludge; good land, ½ acre; medium land, 1 acre; bad land, 2 acres. At the Withington works, Fowler considered it desirable to use an area of at least 1 acre of good sandy soil per 1000 long tons wet sludge.

At Birmingham the total cost of sludge burial, including labor, fuel, interest, sinking fund, etc., was stated in the Fifth Report of the Royal Commission (page 173) at about 8 cts. per long ton of sludge containing 90 per cent. water, at Guildford, about 1 ct. and at Withington about 14 cts.

In large works the preparation of the trenches is quite a problem. Watson (*Proc. Inst. C. E.*, vol. clxxxi) estimated that ¾ mile was prepared daily just before this method of disposal was abandoned.

BURNING SLUDGE

In the United States burning sludge has been attempted at a few places. At Worcester, in 1891, sludge which had been dried in beds until it contained but 50 per cent. water, was readily burned in destructor furnaces, while some containing 72 per cent. water burned at the rate of 2.225 tons in 9 hours unaided by other fuel. At Coney Island, N. Y., for a number of years previous to 1892 sludge was burned in a crematory after it was mixed with sawdust to take up the moisture. (*Engineering News*, Oct. 20, 1892.) In Germany, in works using the lignite process, a dried sludge is obtained which is readily combustible when pressed into briquettes. At Spandau this product finds a market at $1.75 per ton, according to Schmeitzner's "Clarification of Sewage."

The Royal Commission on Sewage Disposal says in its fifth report:

"There have been many attempts to dispose of sewage sludge by burning it, either alone or mixed with house refuse, coal, oil, or resin. Most of these have failed, owing either to the heavy cost of drying the sludge before burning, or to the fact that the attempt was made on wet sludge containing something like 90 per cent. of water. It is practicable, however, to burn pressed cake. At Ealing (Southern Works) Mr. Charles Jones has for some years disposed of pressed sludge (containing about 60 per cent. of water) in this way. At those works the pressed cake is dropped from the presses into trucks, which are then conveyed by a hydraulic lift to the destructor furnaces. There it is mixed with dry house refuse, in the proportion of about 1½ or 2 parts of dry refuse to 1 of pressed cake, and burnt. The steam generated is utilized for lime mixing, for sludge-pressing and the manufacture of artificial paving slabs, and for pumping a portion of the sewage. We have not been able to ascertain the cost of the process, but about 2500 to 3000 tons of pressed cake have been disposed of per annum in this way during the years 1905, 1906, and 1907.

"Reference may also be made to the evidence given by Mr. K. F. Campbell of Huddersfield in regard to an experimental plant. At that place a portion of the sludge obtained by means of chemical precipitation is pressed and, having been tipped into feeding bins at the back of the furnaces, is there mixed with 20 per cent. of coke breeze and fed by hand into the furnaces. The burning plant consists of a 2-cell Horsfall destructor, each cell having a grate area of 30 sq. ft., with a drying hearth at the back; the cells are placed side by side and each is fitted with a patent Silent Steam Blower. The furnaces are so arranged that all the fumes given off by the pressed cake, in drying, have to pass over the hottest part of the fire before they escape to the chimney" (page 177).

"The cost of pressing and burning varies with the prices of the materials (lime, coke breeze, etc.). The following are the average costs for each:

Pressing: cost of pressed cake produced	Per long ton s.	d.	Per 1000 lb.[1] cts.
Labor	1	5.3	15.4
Lime (13s. 3d. per ton)		9.3	8.3
Press cloths		3.7	3.3
Total	2s.	6.3d.	27.0

Burning: cost of sludge cake burned	Per long ton s.	d.	Per 1000 lb. cts.
Coke breeze (7s. 6d. per ton)	1	6.4	16.4
Stokers		7.2	6.4
Mixers		5.3	4.7
Total	2s.	6.9d.	27.5
Total cost of pressing and burning	5s.	1.2d.	54.5

[1] This column has been added by the authors.

"These figures do not include cost of power for pressing on the one hand, but, on the other, no allowance has been made for fuel value or for clinker produced.

"It will be seen that the above process, as carried out on a large experimental scale at Huddersfield, is costly, but probably the whole of the final product can be utilized and the sludge is entirely got rid of" (page 178).

TABLE 113.—RESULTS OF THERMAL ANALYSIS OF CHICAGO STOCK YARDS' SLUDGE

(Calculated to Dry Weight Percentages. Gulick-Henderson Co., Analysts)

Source of sludge	Imhoff tank	Dortmund tank	Chemical precipitation
Fresh sludge:			
Specific gravity	1.02	1.00	....
Moisture, per cent	85.3 – 91.8	95.8 – 96.1	90.1
Nitrogen, per cent	2.73 – 2.96	2.32 – 3.00	2.0
Volatile matter, per cent	56.5 – 72.9	69.5 – 70.6	56.8
Fixed matter, per cent	27.1 – 43.5	29.4 – 30.5	43.3
Ether sol. mat., per cent	4.4 – 6.3	8.6 – 10.6	5.5
B.t.u. per pound	1190	....	1305
Sludge after pressing:			
Specific gravity	....	1.03	1.07
Moisture, per cent	....	80.3	75.6
Nitrogen per cent	....	3.00	2.0
Volatile matter, per cent	....	70.6	56.8
Fixed matter, per cent	....	29.4	43.2
Ether sol. mat., per cent	....	10.6	5.5
B.t.u. per pound	....	....	1795
Sludge after air-drying:			
Specific gravity	1.09	....	1.07
Moisture, per cent	76.1	24.1	75.6
Nitrogen, per cent	2.92	2.56	2.0
Volatile matter, per cent	54.8	41.1	56.8
Fixed matter, per cent	45.2	58.9	43.2
Ether sol. mat., per cent	4.8	....	5.5
B.t.u. per pound	1386	1880	....

At the Philadelphia Experiment Station (Report Sewage Testing Station, 1911) some tests of the calorific value of sludge were made. Sludge was intimately mixed with rice coal in the proportion of equal weights of each. The mixing immediately reduced the water content from 91 to 48.5 per cent. and after 24 hours in a lagoon 12 in. deep, the moisture was reduced to 27.5 per cent. In 9 days the mixture contained 22.5 per cent. moisture. Other tests were made by mixing dried sludge with pea coal just before burning. As burned the mixture contained from 1.15 to 3.87 per cent. moisture.

These experiments showed that while there is some caloric value in sludge, this value is not realized because of the quantity of water evaporated by its burning. They did show, however, that but a small percentage of coal is necessary to incinerate the sludge. No figures of cost are given.

Samples of sludge from the experimental treatment station at the Chicago Stock Yards were submitted to thermal analysis which showed that the calorific value of sludges varied almost directly with the amount of volatile matter (Wisner and Pearse, Report on Industrial Wastes, 1914, page 129). This indicates the desirability of drying fresh sludge as rapidly as possible to obtain the full calorific value, without loss of volatile matter. The results of the tests are given in Table 113.

The use of sludge as a fertilizer is discussed in Chapter XVII.

CHAPTER XIV

CONTACT BEDS

It has been shown in Chapter VI that the changes taking place within a contact bed are very complex, owing to the fact that both aerobic and anaerobic conditions result from the method of operation. The cycle of operation consists of four stages, as explained in that chapter. The design of the beds should aim to make these stages distinct and capable of adjustment as to duration, so that as experience is gained with a plant, the four stages, filling, standing-full, draining and standing-empty, may be varied to meet the local conditions most satisfactorily. Experience with such beds, particularly in Great Britain, shows that careful management is often necessary to avoid serious trouble, but, on the other hand, there are some contact beds, such as the plant at Mansfield, Ohio, which have operated successfully for a number of years without such attention. The feature of contact beds which seems to be most responsible for trouble in some cases is the automatic apparatus for controlling the operation. This automatic apparatus is described in Chapter XVIII.

CONTACT MATERIAL

The aggregates or contact materials best adapted to this process should be given careful consideration before deciding on what materials should be used. Availability and cost, both important factors, are governed primarily by local conditions. There are, however, certain differences in character and physical condition which have an important bearing upon the efficiency of the bed.

Kinds of Aggregate.—Beds have been constructed of various kinds of contact material, cinders, slag, coke, broken stone, pebbles, broken brick, slate and saggers, or waste from pottery plants. Clark and Gage experimented at Lawrence with glass beads and obtained results much inferior to those furnished by beds built of cinders or coke, due, in their opinion, to the smoothness of the surface of the beads. Some have maintained that the greater surface area of substances, actually or in effect porous, such as coke, slag, or cinders, is responsible for their superiority over pebbles or broken stone in producing well-purified effluents. Dr. W. P. Dunbar maintains that a certain quantity of iron favorably distributed through a bed has a beneficial effect and attributes to the

presence of iron in coke its superiority over pumice stone as a contact material. He points out, however, the danger of an excessive quantity of iron, and Clark and Gage found that beds were quickly and completely clogged by excessive quantities of iron distributed through the contact material.

In their experimental work at Lawrence, Clark and Gage found that the intercepted material was quite uniformly distributed throughout the coke beds and suggested that the greater efficiency of these beds in producing good effluents might be due to the effect of the roughness of the surfaces in holding back the suspended matter of the sewage until the organisms had had an opportunity to break up and oxidize it. The smoother materials, like pebbles, offer less resistance to the passage of suspended matter into the lower portions of the bed.

The material should be hard and strong enough to withstand the strains developing within the bed, without breaking and disintegrating. Cinders are generally friable and easily broken into fine particles which fill the interstices in the bed, while clinker is usually hard and able to withstand such action. Coke is so light that it tends to float when the bed is filled, causing more or less disintegration with each such movement. If the bed is built of a soft or friable substance, there will be more or less disintegration into finer particles, and at times of washing the loss of contact material may be a serious item of expense. Furthermore, in such cases the bed is likely to produce a poorly treated effluent and to have its capacity so reduced by clogging as to render its use far from economical. On the other hand, such materials may prove so much cheaper than those more durable, that economy will dictate their use, with enlarged area to offset reduced unit capacity and with provision for frequent washing and consequent replenishing of the quantity of material lost at each washing.

English experience with various kinds of contact material has been summarized by the Royal Commission as follows:

"The experience at Manchester, with regard to contact beds, has been that hard furnace clinker is the best material, being vesicular and yet permanent. Dr. Fowler, however, expressed the opinion that broken Staffordshire bricks, saggers and gravel, if properly sized, should do equally well.

"It will be seen from the evidence that many witnesses have stated that substances like gravel, flints, or broken granite, are suitable as a filtering medium, but that, in most cases, they have expressed a decided preference for the use of those materials which present the roughest and most irregular surfaces, such as clinker, coke and saggers.

"So far as our own experience goes, we should prefer, as a general rule, to use coke or good clinker in a contact bed in preference to other materials.[1]

[1] "Broken 'saggers' might be equally good, but we have had no experience of this material in contact beds."

It is true that there is some evidence to the effect that rather better effluents can be obtained from contact beds of coke than from contact beds of clinker, but we think that this advantage is balanced by the tendency of the lighter material, coke, to shift slightly every time the bed is filled, and therefore to be more liable to disintegration." (Fifth Report, page 66.)

Size of Contact Material.—As one of the principal functions of the aggregate is to afford a large area or surface upon which organisms may make their homes and with which the sewage may come into contact in slowly moving thin films, it naturally follows, theoretically at least, that the finer the contact material the greater will be the degree of oxidation. This is true practically, within certain limits. Besides furnishing a greater extent of contact surface per cubic yard, the fine material tends to hold back the sewage, because of the small voids and narrow spaces through which it has to flow, thus affording a longer time for the action of the physical and biological processes upon the slowly moving films.

On the other hand a fine material, because of the smaller voids, offers a greater opportunity for the retention of the suspended matter and less opportunity for the access of atmospheric oxygen. These are vital objections to the use of too fine a contact material, because the former causes early clogging of the bed and the latter deprives it of an absolute essential to normal action.

The Royal Commission has clearly stated (Fifth Report, page 67) the relation between the size of the contact material and the quantity of suspended matter in applied sewage, in the following words:

"It is important that the material in a contact bed should not be too small, especially if the liquid to be treated contains an appreciable quantity of suspended matter, as some of this suspended matter will undoubtedly find its way into the interstitial spaces and prevent proper drainage of the bed. This was found to be the case with the fine beds at Devizes and Hampton.

"In deciding upon the size of the material to be used, the amount of suspended matter in the liquid to be treated must be considered. As a general rule, the greater the amount of suspended matter in the liquid the larger the material should be.

"With a crude sewage containing 40 parts per 100,000 of suspended matter, the material will probably have to be from 3 in. upward in diameter, and even then sludge will accumulate on the top.

"With a septic tank liquor containing 8 to 10 parts per 100,000 of suspended matter, material of a diameter from ⅜ to ⅝ in. may probably be used effectively; while with a good precipitation liquor containing from 1 to 3 parts of suspended matter, the best results will probably be obtained from material as fine as ¼-in. diameter.

"It is, however, impossible to make any but the most general statement as to the most suitable size of material for contact beds, as, in some cases, there may be special circumstances which affect the question, such as the

character of the suspended matters, or the smoothness of the filtering material. The sizes we have suggested are based on the evidence which we have received and also on our experience of contact beds at the places named in the following statement (Table 114):

TABLE 114.—SIZES OF MATERIAL IN ENGLISH CONTACT BEDS

(From Fifth Report of Royal Commission on Sewage Disposal, page 68)

	Suspended matter in the liquor treated (approx. parts per million)	Nature of material	Size of the material in the contact beds, inches
Crude sewage:			
Hampton	485.	Clinker	Above 4
Leeds	350.	Clinker and coke	Above 3
Newton-le-Willows	300.	Clinker	Top 18 in., ½ to ¼; below, 2 to 1½
Withnell	200.	Clinker	1 to 1½
Maidstone	140.	Clinker	Above ¾
Settled sewage:			
Oswestry	About 200	House coke	1½ to ½
Halton	110.	Clinker and pebbles	1 to 2
Septic tank liquor:			
Leeds	180.	Clinker	⅜ to ⅝
Guildford	160.	Burnt ballast	½ to 3
Hartley Wintney	150.	Clinker	⅛ to ½
Exeter (Main)	About 140	Clinker	½ to 1
Andover	120.	Clinker	⅛ to ½
Exeter (St. Leonards)	85.	Clinker and coke	½
Slaithwaite	80.	Clinker	Top foot, ⅛ to ⅜; below, ⅜ to 1.
Precipitation liquor:			
Calverly	120 to 140	Clinker	¼ to ½
Kingston (experimental beds)	20.	Clinker and coke	Coke bed, ¼ to 1; clinker bed, ¼ to ⅜

Dealing with the effect of the size of contact material, Dunbar states that with clean, rounded Elbe gravel:

"The finer the material the greater is the reduction in the oxygen absorbed; at the beginning of the experiments the reduction was 52.8 per cent. with the finest material (2 to 3 mm., 0.08 to 0.12 in.), and 44.7 per cent. with the coarsest material (10 to 20 mm., 0.39 to 0.79 in.). Correspondingly

the oxygen used up from the air of the beds was larger in amount in the fine beds than in the coarse. In the finest beds it was 62.3 per cent. of the oxygen admitted into the beds and in the coarsest 30.0 per cent. After working for 9 days the values were 94.7 per cent. for the finest beds and 63.8 for the coarsest. The figures representing the production of carbon dioxide show exactly the same variations." ("Principles of Sewage Treatment," page 170.)

Exactly similar experiments carried out with coke of the same sizes as the gravel showed:

"The consumption of oxygen at the beginning was 74.4 per cent. in the finest beds and 42.0 per cent. in the coarsest, higher in both cases than with gravel beds. Carbon dioxide was not present in quantities as large as with gravel beds, because coke absorbs this gas and retains it more than fresh gravel. After 9 days' working, the finest bed consumed 100 per cent. of the oxygen admitted into the bed and the coarsest 59.9 per cent.

"From the above we see that the decomposition process, as well as absorptive action, is more intense in fine material than in coarse, and more pronounced in coke beds than in beds constructed of gravel." (*Ibid.*, page 171.)

Johnson found that as between broken stone and coke, under Columbus conditions, there was but little choice. (Columbus Report, 1905.)

The fineness of the material is limited by the necessity of providing efficient aeration when the bed is emptied, and of providing a size sufficiently great to prevent undue clogging, this latter requirement being partly governed by the composition of the particular sewage.

In 1905 Johnson adopted sizes ranging from ¼–1 in. to ½–1½ in. for the experimental contact beds at Columbus, basing his action upon the results of the studies at Lawrence.

At Plainfield, N. J., the primary filters were originally filled with ¼ to 1-in. material. They became badly clogged in about 4 years. In 1905 some new beds were built in which the size of material ranged from ½ to 1 in., and still more recently it was recommended that the size of material in new beds should be 1 to 2 in. ("Sewage Disposal," Fuller, page 679.)

In other American cities contact beds have been constructed as follows: Sturgis, Mich., ¼ to 1½-in., limestone; Great Lakes, Ill., ¼ to 6 in., granite; Bellefontaine, Ohio, ½ to 1 in.; Galion, Ohio, ⅛ to ¾ in. crushed and screened cinders; Madison-Chatham, N. J., 1 to 2½ in.; Alliance, Ohio, ⅛ to ¾ in. cinders; Grand Canyon, Ariz., ¼ to 1 in. volcanic cinders.

DEPTH OF BEDS

This is largely an engineering problem, being limited on biological grounds only by the requirements for satisfactory aeration, filling and

emptying which have been found to be desirable. The Royal Commission on Sewage Disposal (Fifth Report, page 52) found that within ordinary limits the depth of a contact bed makes practically no difference in its efficiency per cubic yard. It recommended that beds be not more than 6 ft. or less than 2 ft. 6 in. deep. With regard to the minimum and maximum depth, it said:

"In order that a contact bed may drain properly, it is necessary to have a few inches of large material on the floor of the bed, and this material, with the drains themselves, would occupy about 6 in. of depth. The sewage which remains among this coarse material, or in the drain spaces, during the time when the bed is full is not so well purified as the liquor which is standing among the finer material in the upper part of the bed, and it is found that the first flush from a contact bed, which comes from the lowest part of the bed, is usually considerably worse than the main bulk of the effluent. As a general rule, we doubt whether it would be wise to allow this less efficient portion of the bed to form more than one-fifth of the whole depth, and for this reason we suggest that the minimum depth should usually be not less than 2 ft. 6 in.

"As regards the maximum depth of contact beds we may observe—

"(a) That an increase in depth puts a greater weight upon the filtering material in the lower part of the bed, and may cause disintegration.

"(b) That the difficulty of digging and handling the material, when washing is necessary, is greater with deep than with shallow beds, and that the operation would require a greater amount of labor and would result in more breaking down of the material during the process" (page 53).

It has been the usual custom to place a layer of large stones, usually not less than 6 in. in diameter, upon the floor of contact beds to provide better drainage than that afforded by the relatively small stones or filtering material composing the main body of the bed. This constitutes practically waste depth, so far as contact surface is concerned, and still better drainage could undoubtedly be afforded by a floor system similar to those now commonly used with trickling filters, which would also furnish a better opportunity for the suspended matter of the sewage temporarily held in the bed to be washed out. In the report on the operation of the contact beds at Manchester, England, for 1905–6, it is stated that loss of capacity was often found to be due more to imperfect drainage than to actual accumulation of solid matter. If the surface area of the contact material is a feature of great importance, as has generally been conceded, little can be said in favor of the large stones, as they present a relatively small area and afford a poor underdrainage system. It is probable, also, that in the former respect the draining floors would not be materially less effective than the large stones, and it is probable that they could be made enough thinner than the ordinary floor plus the large stones so that the depth of fine material might be

increased enough to furnish a contact surface equal to that of the coarse stones. The chief objection to this plan would probably lie in the larger storage capacity of the draining floor, although this might be overcome in part by filling this space with effluent from an adjacent bed. As pointed out by the Royal Commission, the first water leaving the contact bed is inferior to that which follows. With a large storage capacity in the floor drains, the proportion of this inferior effluent might be greater than where the coarse stones are used for draining purposes.

The quantity of sewage which can be successfully treated by a given contact material depends upon the cubic space afforded by the voids in the material. It therefore follows that by increasing the depth of the bed the quantity of sewage which can be treated will be correspondingly increased, thus reducing the area required and the expense of floors, walls and appurtenances. Within reasonable limits economy may be effected in this manner in some cases.

Of 16 beds examined, the Royal Commission found that 11 were between 4 and 5 ft. deep. Experiments at Lawrence indicated that the depth had very little effect on the biological activity of contact beds. (Report Mass. St. Bd. Health, 1908, page 445.)

The Plainfield, N. J., beds are 5 ft. and those at Mansfield, Ohio, 4 ft. 9 in. in depth. The depths of other American beds are as follows: Sturgis, Mich., 2¾ ft.; Great Lakes, Ill., 8 ft.; Bellefontaine, Ohio, 4½ ft.; Galion, Ohio, 4½ ft.; Madison-Chatham, N. J., 4.2 ft.; Alliance, Ohio, 5 ft.; Grand Canyon, Ariz., 4 ft.

RATE OF OPERATION

Contact beds are generally dosed by completely filling the voids at a single operation. If a 1-acre bed 5 ft. in depth is composed of contact material having 30 per cent. voids, one dose would consist of 488,700 gal. of sewage, and the rate of operation would depend upon the number of fillings per day. It is difficult, however, to determine the proper number of fillings per day, as this depends upon the quality and strength of the sewage, the quantity and character of the suspended matter applied, the size and character of the contact material, the extent to which the material is clogged, and the character of effluent which is to be produced. The stronger the sewage and the greater the quantity of suspended matter in it, the smaller will be the quantity which the filter can receive and convert into an effluent of good quality. A fine contact material will be unable to treat as great a quantity as a medium-size or coarse material, although for a time the quality of the effluent produced by the first may be decidedly better. A clogged filtering material passes sewage slowly and the air cannot penetrate it freely, both of which conditions limit the capacity of the bed to relatively small quantities of sew-

age. If the effluent need not be of the highest purity attainable, the quantity which may be applied to the filter may be somewhat greater than if it must be as well treated as possible.

It was found at Lawrence that when beds were filled twice daily the effluents were inferior to those produced when the beds were filled but once each day, except in two cases where, without apparent loss of efficiency, the beds were filled twice daily during a period of slightly more than 2 months during the first year of their operation.

Increasing the number of daily fillings obviously greatly increases the load put upon the bed, a change from 1 to 2 fillings increasing the load 100 per cent. When a bed is relatively new and free from clogging material, its water capacity is so great that ordinarily the load put upon it by 1 filling per day of strong sewage is as great as it is capable of carrying, while producing an effluent of good quality. After the bed becomes older and is more or less clogged with sewage matter and fine particles broken off from the contact material, its water capacity becomes less, but its oxidizing ability is correspondingly curtailed so that generally 1 filling a day represents all the work it is capable of doing well.

Experience has shown that contact beds must eventually be cleaned, but it is probable that the rate of application can be so adjusted as to procure a satisfactory effluent and postpone the time of cleaning for a period of at least 5 years, and with care and reasonable preliminary treatment by sedimentation for a period of 10 years. Every effort should be made to reduce as far as practicable the quantity of suspended matter entering the bed if its life is to be prolonged to the limit, its effluent kept high grade and its capacity maintained as great as possible. It is, therefore, probably advisable in most cases to remove, by sedimentation, as much as practicable of the suspended matter, not only in the sewage applied to the primary beds but also in the effluent applied to the secondary beds.

Periods of rest must be taken into account in computing the rate of operation of contact beds. At Lawrence the beds were filled on 6 days per week and in general were rested 1 week in 6. The filter, therefore, was not dosed on 12 days in 6 weeks or 28.6 per cent. of the total time, and the actual rate of operation was 28.6 per cent. less than the nominal rate based upon the number of fillings per day.

Two experimental beds at Lawrence were operated without cleaning for over 9 and 11 years respectively. No. 176 received raw sewage for about 6 years, after which settled sewage was applied to it until it became clogged. No. 175 received sewage which had been strained through coke or coal strainers. The beds were ¼ to 1-in. coke.

The actual average rate of operation of bed 175, taking into account rest periods and weighting the average by allowing for part of 1901 when

the filter was not operated, was 369,000 gal. per acre daily. Similarly, the actual rate for bed 176 was 360,000 gal. per acre daily. If it is assumed that the average person contributes 15 grams of unoxidized nitrogen to the sewage daily, 1 acre of bed 5 ft. deep like No. 175 would be adequate for each 3910 persons, and 1 acre 5 ft. deep like No. 176 would be adequate for 4490 persons. It should be added that bed 175 removed 66 per cent. of the unoxidized nitrogen and bed 176 only 60 per cent.

In the 1899 report of Latham, Frankland and Perkin on improved sewage treatment works at Manchester, England, it was proposed to use beds 3 ft. 4 in. deep, and fill them four times every 24 hours. On this basis an acre was estimated to have a capacity of 600,000 U. S. gal., allowing a rest of 1 day in 7. Before carrying out the project on a large scale, 2 half-acre beds were built for experimental purposes. The contact material was 3 ft. 1 in. of clinker rejected by a 3/16-in. screen and a top layer of screenings 3 in. thick. The beds were started with 1 filling a day and gave, in 1900–01, "a strictly non-putrefactive effluent." Other similar beds were constructed a little later, and by July, 1902, the areas in service longest were treating about 480,000 U. S. gal. per acre daily. Meanwhile a permanent plant was placed under construction, consisting of 92 half-acre primary beds, to receive septic-tank effluent, and 27 acres of secondary beds to receive the effluent from the primary beds. In 1904-5, before the secondary beds were finished, the official report indicated that putrefaction took place in the effluent of the primary beds, but it was due to the suspended matter present. The following year "the results show a slight falling off as compared with previous years." The average rate for the year was 649,000 U. S. gal. per acre per day (including all periods of rest), more than originally contemplated, and renewal of material became necessary in the small channels formed in the surface of the bed to assist in the distribution of the sewage. In 1906–7 the primary beds were operated at an average rate of 574,000 gal. per acre daily, and a permanent plant for washing material taken from the beds was in constant operation. At that time, it was estimated from the experience to date that a bed could receive nearly 2700 fillings before the aggregate would need washing. During 1909–10 the present secondary beds were put under construction; during that year the primary beds were worked at the rate of 713,000 gal. per acre daily. The primary bed rates in the years ending March 25, 1911 to 1914, inclusive, were 661,000, 576,000, 567,600 and 612,000 U. S. gal. per acre per day, part of which was treated on secondary beds during the last year of this period, at the rate of 18 fillings per week. By this date a second renewal of the material had been necessary in a number of cases, the life of the bed after the first renewal ranging from 4 to 5.4 years. The material in the primary beds was worked at a rate of 114 U. S. gal. per

cubic yard, while the rate was only 106 gal. the previous year. The number of fillings of beds between the first and second renewals ranged from 4786 to 6404. The water capacity of a half-acre bed ranged from 216,000 U. S. gal. when new to 108,000 gal. after 2250 fillings, on an average. By the end of this year, the annual cost of treatment was 24.6 cts. per capita for operation and maintenance and 23.8 cts. for interest and sinking fund, a total of 48.4 cts. per capita, and the committee in charge of the plant officially announced that the effluent was unsatisfactory and reconstruction and alterations were contemplated.

Early in 1911, 8 contact beds were put in service at the Lawrence Experiment Station to determine the effect of operating them by different methods. Each bed is 2.2 sq. ft. or 0.00005 acre in area, 33 in. deep, filled with ¼ to 1¼-in. soft-coal cinders and dosed with settled sewage. The results up to the close of 1913 are given in Table 115.

The effect of the standing-full period is clearly shown in this table. The fact that the nitrates follow the same curve as the ammonias was explained by Clark and Gage as follows:

"The reactions taking place within contact filters are somewhat different from those in filters of other types, in the fact that nitrates which have been formed during the resting or oxidizing cycle of the filter may be again reduced and yield oxygen for the oxidation of other organic matters during the time the filter is filled with sewage. This is undoubtedly the explanation of the differences in the amount of nitrates in these effluents." (Report Mass. St. Bd. Health, 1912, page 306.)

Attention was called by Clark and Gage to the difference in stability between these results and those with filters in which oxidation plays a more important part. The results of stability tests of the effluents of contact beds do not correspond with the nitrates. About half the samples from the bed showing the highest nitrates were putrescible, while no putrescible samples were obtained during 1913 from the filter in which the sewage was held for 8 hours and in which any nitrates that had been formed during the resting stage had been again reduced during the long period the sewage was standing within the filter.

Rate of Filling and Draining.—As these two operations have no biological significance it is generally held that the more quickly they take place the better, as long as the action is not so rapid as to disturb the materials or the film surrounding them in the bed.

Standing Full.—This period should not be long enough to allow anaerobic decomposition to an appreciable extent. In some cases Clark and Gage (Report Mass. St. Bd. Health, 1908, page 445) found no marked difference when the period of contact varied from 0 to 5 hours, but, on the whole, periods exceeding 2 hours were productive of inferior effluents.

Johnson (Columbus, 1905) decided that under local conditions even 1 hour was too long a period for the most satisfactory results.

TABLE 115.—TESTS OF SIMILAR FILTERS UNDER DIFFERENT OPERATING CONDITIONS

(Report Massachusetts State Board of Health, 1913)

Filter No.	Filled from	Time of filling, minutes	Standing full time, hours	Times filled daily	Dose, gallons per acre daily	Effluent, parts per 1,000,000				Samples putrescible, per cent.
						Ammonia		Nitrates	Oxygen consumed	
						Free	Albuminoid			
421	Top	2	8	1	383,800 in 1911	19.89	2.24	1.0	16.2	......
					377,100 in 1912	15.69	2.50	0.9	15.8	......
					279,300 in 1913	14.96	2.62	0.1	16.6	0.0
422	Top	2	4	1	376,500 in 1911	19.61	2.48	1.4	17.0	......
					375,400 in 1912	17.06	2.66	2.6	17.0	......
					277,700 in 1913	16.79	2.68	3.7	16.9	5.5
423	Top	2	2	1	391,000 in 1911	21.72	2.55	1.8	18.5	......
					404,100 in 1912	18.48	2.92	2.6	19.2	......
					312,300 in 1913	18.31	2.97	4.1	19.8	44.4
424	Top	2	1	1	388,400 in 1911	24.33	2.70	2.1	19.9	......
					401,000 in 1912	20.33	3.23	2.9	21.3	......
					304,900 in 1913	18.67	3.00	5.5	21.4	55.6
425	Top	2	1	2	671,800 in 1911	22.22	2.81	1.4	20.9	......
					765,000 in 1912	21.07	3.59	3.3	22.1	......
					548,700 in 1913	18.72	3.45	5.6	22.6	55.6
426	Top	2	1	3	894,800 in 1911	21.49	2.71	1.7	20.4	......
					1,004,700 in 1912	20.07	3.22	1.9	21.9	......
					599,000 in 1913	16.92	3.09	4.4	21.8	5.6
427	Bottom	2	1	1	396,400 in 1911	24.87	2.79	0.7	21.7	......
					426,100 in 1912	23.30	3.37	1.3	22.7	......
					324,200 in 1913	18.79	3.09	3.4	23.4	38.8
428	Top[1]	60	1	1	387,300 in 1911	19.95	2.41	3.2	18.3	......
					397,600 in 1912	18.50	2.69	7.5	18.7	......
					304,100 in 1913	17.90	2.97	8.9	19.8	33.3

[1] Dose applied by tipping basin emptying into perforated pan 1 ft. above surface of bed, the sewage dripping through the perforations.

The Royal Commission on Sewage Disposal stated in its Fifth Report:

"The evidence shows that 2 hours' contact and 4 hours' rest have generally been found to give the best results in practical working, where the beds are filled three times a day, but no rule can be laid down which is of universal application.

"As regards this question, Dr. Fowler stated in reference to his Manchester experience that the periods depend almost entirely on the age of the bed and

the dilution of the sewage. In the initial stages of working, long contacts (*e.g.*, 24 hours) have been found advantageous, their effect being to facilitate the formation of a slimy layer of colloidal matter on the medium, in which the real biological action begins. As this layer increases, the 'absorptive' effect increases also, and less time of contact is required. Moreover, a greater quantity of water is held up in a bed, and the quantity of 'drainage' increases, till with a bed in long use a very short contact, say a quarter of an hour, is all that is required. With increase in dilution of the tank effluent, due, *e.g.*, to storm water, the period of contact may be reduced to a minimum. In general, after the bed is once 'mature,' the period of rest is more important than the period of contact, and the total time occupied in 24 hours in filling, standing full, and emptying should not exceed the total of the period of rest. Thus, with frequent fillings, the time of contact should be shortened. If the bed takes long to fill, the extra time in filling should be taken from the time of contact. The above also applies to secondary beds, it being understood that, as they deal with an effluent containing less impurity, they can be worked more rapidly than primary beds.

"Unless the circumstances are exceptional, we think that in designing a contact bed scheme a contact of 2 hours' duration should be provided for, leaving subsequent experience to show whether this could not be shortened to some extent. The period of contact should be fairly regular, as sudden variations may, we think, have a disturbing effect upon the agents of purification. We have noticed, for instance, in beds generally given 2 hours' contact, that if the sewage is occasionally held up for a much longer time, say 5 or 6 hours, great numbers of worms are driven to the surface" (page 54).

At Plainfield, N. J., the beds have been emptied as soon as filled (*Engineering News*, July 1, 1911), although this extreme condition was due to lack of capacity.

Standing Empty.—The consensus of opinion is that the most important biological purification of the organic matter takes place while the bed is standing empty, and that this period should be made as long as possible, taking into account the necessary time for filling, emptying and standing full.

MULTIPLE CONTACT

The inability of the single-contact bed operated under many conditions to produce a highly purified effluent has led to applying the effluent from one bed to a second bed, and in a few cases the effluent from the second bed to a third bed.

Dunbar and Thumm, 1902, found that the effluent from double-contact beds, where the primary bed was filled six times a day and the secondary bed three times a day, was equal to that of a single-contact bed filled twice a day.

Johnson found that practically double the rates permissible with primary beds could be employed with secondary beds with reasonable results as to the quality of the effluent. He stated:

"By employing a double contact, the net rates per acre for both treatments could probably be increased over those possible in the case of single-contact filters, obtaining thereby equally satisfactory results. Such increases appear to be in the neighborhood of 20 per cent., according to local data." (Columbus Report, 1905, page 273.)

At Manchester, early experiments with first- and second-contact beds, and with single-contact beds operated at one-half the rate, clearly showed that the double contact was more effective than the single contact at one-half the rate, as shown in Table 116.

TABLE 116.—RESULTS OF DOUBLE- AND SINGLE-CONTACT TREATMENT AT MANCHESTER, ENGLAND

(Parts per 1,000,000; from "Purification of Boston Sewage," Winslow and Phelps, 1906, page 68)

	Nitrogen as			Oxygen consumed in 4 hours at 80°F.
	Free ammonia	Albuminoid ammonia	Nitrates and nitrites	
Septic effluent	25.8	2.5	....	70.0
First contact	14.5	1.2	0.5	22.0
Second contact	4.1	0.5	8.5	6.9
Septic effluent	31.0	3.5	....	80.0
Single contact (one-half rate).	13.3	1.3	4.3	16.0

Among the best results obtained at Lawrence, Mass., by pairs of primary and secondary experimental contact beds, were those from beds 137 and 164. Unfortunately these beds were operated together for but a short time, the latter bed being in use only about 5½ months, a period entirely too short to indicate the length of time which it could be operated without undue clogging.

The data relative to these 2 beds may be grouped as follows:

Bed 137. Depth, 5½ ft. Broken stone, 1 to 1½ in.
Untreated sewage applied:
First year, filled once daily.
Second year, twice daily.
Few months, three times daily.
Filter clogged in about 2 years.

Bed 164. Depth, 4½ ft. Coke free from dust, all passing ¼-in. screen.
Effluent from filter 137 applied:
One filling daily.
Duration of operation, Jan. 10, 1901, to June 30, 1901, 5½ months.

While the quantity of raw sewage applied to bed 137, expressed in gallons per acre per day, was greater than the quantity of effluent applied to bed 164, the quantities per cubic yard, allowing for the difference in depth of beds, were 77.0 gal. and 90.1 gal. per day, respectively. The average quantity applied to both beds was 83.55 gal. per day, equivalent to 741,000 gal. per acre per day applied to a bed 5½ ft. in depth.

The effluent produced by bed 137 was much inferior to that produced by a number of other single-contact beds, but that it was greatly improved by second-contact treatment in bed 164 is evident from the average analyses given in Table 117.

TABLE 117.—AVERAGE ANALYSES OF EFFLUENT FROM DOUBLE-CONTACT SYSTEM, LAWRENCE EXPERIMENT STATION

(Report Mass. State Board of Health, 1908, page 423)

(Parts per 1,000,000)

	Free ammonia	Total albuminoid ammonia	Dissolved albuminoid ammonia	Nitrogen: As nitrates	Nitrogen: As nitrites	Oxygen consumed	Chlorine	Bacteria
Effluent from bed 137.	32.4	3.2	1.9	2.3	0.006	21.7	91.8	502,000
Effluent from bed 164.	10.5	1.4	1.0	21.7	0.162	9.4	82.4	210,300

It is unfortunate that these beds were not continued in operation for a much longer period of time. In order to have accomplished this, it would undoubtedly have been necessary to have reduced materially the load upon bed 137, and it is probable also that a smaller load upon the secondary bed would have been necessary. While these experiments are interesting as showing what it is possible to accomplish, they can hardly be regarded as showing what it is practicable to accomplish, for the rate of operation of the primary bed was apparently far in excess of that at which it could have been operated for a period of years sufficient to have made it an economic success, and the contact material in the secondary bed was so fine that it could hardly be expected to continue in successful operation for a long time.

Another interesting pair of double-contact beds used at the Lawrence Experiment Station are Nos. 221 and 237. The essential data relating to the construction and operation of these filters are given in Table 118.

The work of this pair of beds is of particular interest because the primary bed received raw sewage during most of its life, and because the effluent from the secondary bed was at all times well nitrified and stable. It was not, however, quite as good as the effluent from bed 175 when doing its best work. Bed 221 was seriously overloaded with sus-

TABLE 118.—DOUBLE-CONTACT BEDS 221 AND 237, LAWRENCE EXPERIMENT STATION

(Report Mass. State Board of Health, 1908)

	Primary bed 221	Secondary bed 237
Period of operation	July 7, 1903–Oct. 27, 1908	Jan. 1, 1904–Oct. 28, 1908
Contact material	Broken stone	Clinker
Size, inches	25 per cent., ¼ to ½; 75 per cent.; ½ to 1.	¾ to 1¾
Depth	3½ ft.	5 ft.
Area	8.7 sq. ft.	2.2 sq. ft.
Liquid applied	July 7, 1903-Aug. 15, 1906, station sewage	Effluent from bed 221
	Aug. 15, 1906-Oct. 27, 1908, settled sewage	Effluent from bed 221
Contact period	Two hours	Two hours
Fillings per day	July 7, 1903–Dec. 1, 1904, one	Two
	Dec. 1, 1904–Oct. 27, 1908, two	Two
Rest periods	1 day in 7, throughout test	1 day in 7, throughout test
	July 8, 1907–Oct. 27, 1908, 1 week in 6	July 8, 1907–Oct. 28, 1908, 1 week in 6
Average dose, gal. per acre daily	483,000 nominal; 402,000 without deducting rest periods	1,169,000 nominal; 950,000 without deducting rest periods
Average dose, gal. per cubic yard daily	85 nominal or 71 for entire time without deducting rest periods	144 nominal or 118 for entire time without deducting rest periods
Applied nitrogen: 1904	17.5 grams per cu. yd.	....
1905	13.6 grams per cu. yd.	....
1906	11.3 grams per cu. yd.	....
1907	9.3 grams per cu. yd.	....
1908	11.0 grams per cu. yd.	....
Average	12.58 grams per cu. yd.	....
Equivalent number of persons served: 1904	9420 per acre of bed 5 ft. deep	11,200[1] per acre of bed 5 ft. deep
1905	7320 per acre of bed 5 ft. deep	6,250 per acre of bed 5 ft. deep
1906	6080 per acre of bed 5 ft. deep	14,700 per acre of bed 5 ft. deep
1907	5000 per acre of bed 5 ft. deep	12,900 per acre of bed 5 ft. deep
1908	5930 per acre of bed 5 ft. deep	10,500 per acre of bed 5 ft. deep
Average	6780 per acre of bed 5 ft. deep	11,100 per acre of bed 5 ft. deep

[1] These estimates are based on the sewage applied to bed 221, assuming 15 grams unoxidized nitrogen per capita per day.

pended matter and was discontinued because it became badly clogged. Bed 237 became somewhat clogged during the 3½ years when it was operated without prolonged rest periods, but the open space increased practically to its original volume within 6 months after the filter was allowed 1 week in 6 for rest.

At some places triple-contact beds have been investigated, but as considerable head is necessary to operate such a plant, the additional cost is generally greater than is justified by the additional purification attainable.

Under many conditions the most satisfactory arrangement appears to be a set of double-contact beds, the first of coarse material which will retain most of the solid matter, and the second of finer material for further purification.

MATURING OF BEDS

The purification effected improves for a considerable time after a bed is first put in operation. Dunbar ("Principles of Sewage Treatment," page 175) says:

"During the first few weeks of operation the purification effected by biological filters increases from day to day. This is explained as follows:

"(1) The absorbed substances are deposited on the separate particles of the material composing the filter.

"(2) The suspended matters which gain access to the filter and the matters precipitated on the material by absorption are not completely decomposed, but only deprived of their easily decomposable constituents. A not inconsiderable portion, similar in character to humus, remains on the surface of the gravel or clinker, and this increases the absorptive power of the filter. The gelatinous nature of this coating, to which in the preceding chapter we attributed the character of a surface film, is increased by the micro-organisms and higher forms of life, both animal and vegetable, which soon begin to inhabit the filter. In contact beds the higher forms of life predominate near the surface, where they are able to obtain sufficient oxygen and where they obtain nourishment from the accumulating sludge, which they break up and loosen to a remarkable extent."

Fowler gave the following testimony in 1905 before the Royal Commission on Sewage Disposal (Fifth Report, Appendix 1, page 360):

"Half-acre beds which, after several weeks' working, failed to give a non-putrefactive effluent when only filled six times a week, 2 hours' contact being allowed, immediately gave a non-putrefactive effluent when the period of contact was increased to 12 hours. With Manchester sewage, therefore, the best method of starting new beds appears to be to fill them not more than once a day, giving at least 12 hours' contact, this period of contact being gradually reduced as the beds become more mature. With beds which have been in use for several years, the total period of filling, standing full and emptying need not exceed 2 hours."

Clowes and Houston from experiments with London sewage ("Experimental Bacterial Treatment of London Sewage") came to the following conclusion in 1904:

"In consequence of this bed having received 2 fillings daily before it was properly matured, it became foul, and was, therefore, unable to deal with the sewage in a satisfactory manner. It is now known that when a new bacteria-bed is first started, the work imposed upon it must be very slight until the bacteria in the bed have multiplied sufficiently to cover the surfaces of the material of which the bed is composed.

"Most observers who have had any practical experience of the so-called biological treatment of sewage are agreed as to the necessity of treating new bacteria-beds, until they have become thoroughly matured, with small but gradually increasing doses of raw sewage. Further, it is generally believed that a mature bed is one which has become by a natural process of selection charged with the special bacteria concerned in the work of purification" (page 126).

LOSS OF CAPACITY

As soon as a contact bed is put in operation it begins to lose capacity. This may continue until the voids are completely clogged and the bed inoperative, or it may stop after reaching a mature condition, depending on circumstances of design and operation.

The water capacity of a bed depends partly upon the character of the contact medium. If the particles are substantially uniform in size and spherical in shape, the open space will be much greater than if the materials vary in size and are very irregular in shape.

At Lawrence, the initial open space was found to vary in the several beds from 32 to 67 per cent., and there was considerable variation in the open space in the beds constructed of similar material. It was found at Lawrence that "in practically all of the filters, the reduction in open space was continuous when the method of operation was not modified" (1908 report, page 437). In bed 175, operated with strained sewage, the open space decreased from 59 to 47 per cent. during the first 2 years, and at the end of the seventh year it had been reduced to 33 per cent. of the original space. The systematic resting of the beds resulted in an increase of the open space. "In filter 175 the volume of sewage which could be treated in the filter daily was increased 16 per cent. by a rest of 1 week. In filter 176 the increase after resting was 23 per cent. and in filter No. 221, 41 per cent. (1908 report, page 440).

Fig. 126, prepared from illustrations in the report of the Massachusetts State Board of Health for 1908, illustrates the changes and the amount of decrease in the open space from year to year from the beginning of operation of the several beds at the Lawrence Experiment Station.

Clark and Gage found that the clogging material within the beds varied in location materially from year to year, and also found that in

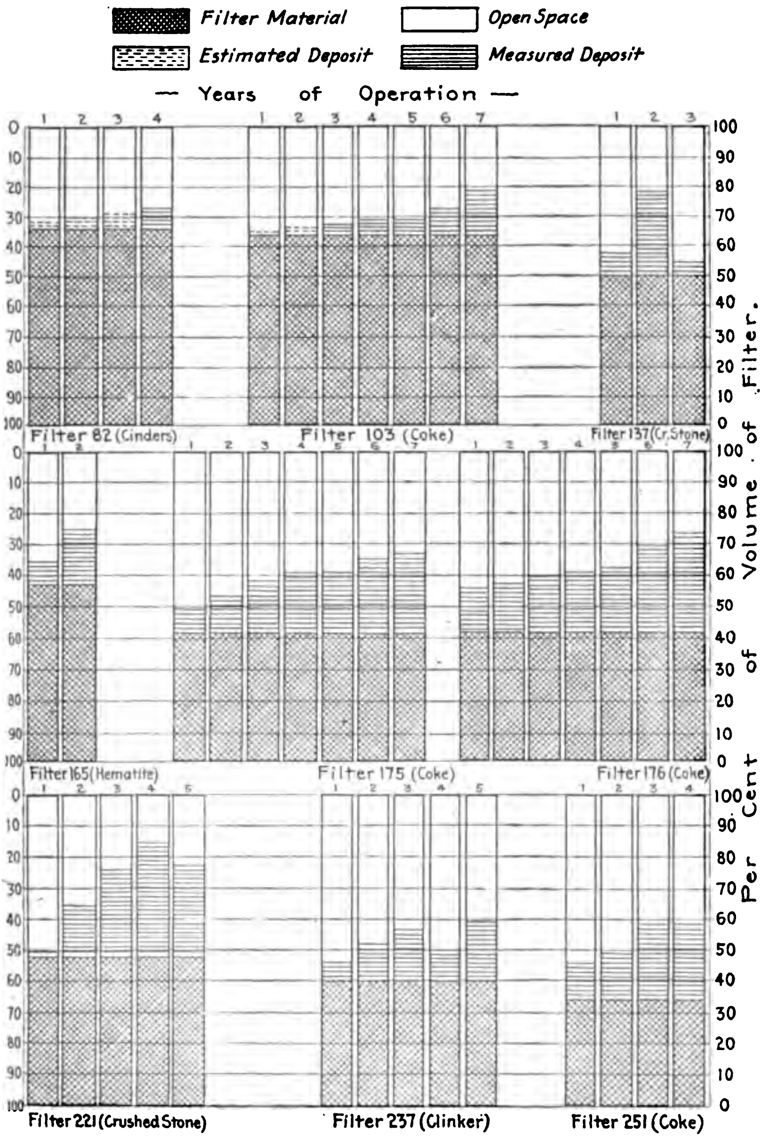


FIG. 126.—Clogging of contact beds, Lawrence Experiment Station.

some of the beds most of the clogging matter was at the bottom, while in others it was largely near the top. During the earlier years of the operation of bed 175, which received strained sewage, the accumulation of stored matter appeared to be practically uniform throughout the contact material. At a later date, however, the bottom of the bed became quite badly clogged, although at the time there appeared to be no reduction in the open space of the upper layers. At a later date, the clogging of the lower layers had been quite largely eliminated, although the upper layers had become somewhat clogged, and still later the lower layers again became badly clogged.

The Royal Commission on Sewage Disposal summarized the causes of loss of capacity as follows:

"The loss of capacity of contact beds is undoubtedly a serious drawback to their efficiency. The evidence shows that the loss chiefly depends upon the following factors:

"(1) Disintegration of the filtering material.
"(2) Consolidation of the filtering material.
"(3) Deposition of colloidal matter.
"(4) Growth of organisms.
"(5) The volume of liquid passed on to the bed.
"(6) Insufficient rest.
"(7) Inefficient drainage.
"(8) The amount of suspended matter in the liquid passed on to the bed." (Fifth Rept., page 56.)

Disintegration.—Where beds are constructed of friable material, such as coke or cinders, disintegration may be serious. At Leeds, after 1 year's use of coke, originally 3 in. or more in diameter, 45 per cent. was found to pass a 1½-in. mesh.

At Manchester, contact bed 10, composed of clinker and cinders larger than 3/16 in., received 2578 fillings. Afterward 24 per cent. passed through a ¼-in. mesh, two-thirds of this being between ⅛ and ¼ in. in size. Similar results were found at Newton-le-Willows, Andover and Oswestry.

Consolidation.—"Consolidation of the material usually accompanies its disintegration, the small broken pieces becoming washed into the interspaces of the bed. The same thing takes place where the separate fragments of material in a bed are of unequal size.

"In the Leeds experiments it was found that contact bed No. 8, treating septic-tank liquor, had its capacity reduced within 6 months from 29,500 gal. to 10,700 gal., and this notwithstanding that the material below the upper 2 in. was still comparatively clean. The bed was rested for 6 weeks and the material turned over, with the result that on restarting the capacity was found to be 26,900 gal. The material in this bed was furnace clinker of ⅛ to ⅜ in. diameter.

"Much the same result was noticed during the filtration of lime-precipitated liquor at Leeds in a contact bed. The original water capacity of this bed, on March 24, 1899, was 55,700 gal.; on October 20, 1899, after rather over 7 months' working at 3 fillings per 24 hours, it had fallen to 21,600 gal. Here again the material (clinker of ⅝ to 1 in. diameter) below the upper few inches was found to be fairly clean, though much consolidated.

"It was concluded that the loss of capacity in both cases was due very largely to consolidation of fine material of uneven size, and our own observations of these beds bear this out.

"In another case—Hampton—where very much finer material was used, the same thing was observed. Tertiary bed No. 2, composed of boiler furnace clinker under ¼ in. diameter, including dust, after being in use for 5 years, became so consolidated that much difficulty was experienced in getting the secondary bed effluent into it. The bed was then simply turned over, and a considerable increase of capacity resulted. From the appearance of the material before it was turned over (which, although clean, was closely bound together), it was evident that the greater part of the loss of capacity was due to the consolidation of the material rather than to clogging from suspended matter.

"Beds of very fine material, as in this case, are undoubtedly more liable to consolidation than beds of medium-sized or large material. In some beds of the same kind at Devizes, we observed that the fine material (which also included dust), although fairly clean, was so closely bound together that the liquid of one filling had not time to run away before the next filling was due, with the result that putrefaction set up in the body of the bed." (Royal Commission on Sewage Disposal, Fifth Rept., page 57.)

Growth of Organisms.—In regard to this point Dr. Fowler stated:

"'This (growth of organisms) is at once the cause of increased efficiency in the bed and of loss of capacity. On examining the material of a contact bed in active condition, every piece is seen to be coated over with a slimy growth. If this is removed it soon dries to a stiff jelly, which can be cut with a knife. Under the microscope masses of bacteria and zooglœa will be found to be present. If placed in a tube containing air, and connected with a manometer, the jelly will rapidly absorb all the oxygen and produce carbon dioxide.

"'This action will sometimes produce a vacuum of several inches of mercury. There is little need, therefore, to force air into a bed, as the natural interchange of gases which takes place is sufficient for adequate aeration. As a matter of fact, there is always a large amount of oxygen to be found at the bottom of a bed in good condition. The behavior of the bacterial jelly appears to afford the clue to the successful working of bacteria beds. By working them at a high speed—*i.e.*, filling them frequently in the day, without long periods of rest—the effluent may remain good, but the bacterial growth so rapidly increases that the bed becomes too spongy, and will not allow the water to drain away. Here, too, is the explanation of the fact that, within certain limits, decrease of capacity is accompanied by increase of efficiency.

"'This decrease of capacity may, however, become so great as to out-

weigh the advantage of increased efficiency. A long period, say 1 or 2 weeks' rest, must then be given to the bed.

"'These rests should not exceed a fortnight at most, as the bed then tends to dry up, and the activity of the organisms diminishes.

"'It is, therefore, important that the decrease of capacity should not be allowed to become excessive before resting, or it may not be possible to completely recover the loss during the time of rest.'" (*Ibid.*, page 58.)

"Leeds.—During the experiments upon the filtration of Leeds crude sewage, the primary bed, which started on October 2, 1897, with a water capacity of 83,000 gal., had been reduced by February 2, 1898, to 45,000 gal. capacity. After a fortnight's rest, its capacity was again measured and found to be 56,000 gal., or an increase of 11,000 gal."

"A similar increase of capacity followed subsequent rests.

"Hampton.—At Hampton, No. 2 primary bed started in the early part of 1899 with an estimated water capacity of approximately 43,500 gal., and was found on February 3, 1903, to have a capacity of only 8637 gal. On September 23, 1904, after a fortnight's rest, its capacity had risen to 12,965 gal." (*Ibid.*, page 59.)

Suspended Matter in Liquid.—"Where the greater part of the suspended matter in the incoming liquid is allowed to find its way into the body of the material, the loss of capacity tends to vary in direct proportion to the amount of suspended solids in the sewage or tank liquor which is being treated. If very fine material is used in a bed, or if a bed of medium-sized material is coated with a layer of very fine material, a large proportion of the suspended matter is caught upon the surface." (*Ibid.*, page 59.)

"There is a large amount of experience showing that primary contact beds lose their capacity at a greater rate than secondary beds, and we think there can be no doubt that this is largely due to the fact that the primary bed usually receives considerably more suspended matter. At the same time the facts that the suspended matter in a primary bed effluent is of a somewhat different character to the suspended matter in a tank liquor or a crude sewage and that there is less colloidal matter in a primary bed effluent would, to a lesser extent, account for the differences.

"The following table (Table 119) shows the loss of capacity, per 1,000,000 gallons of liquor treated, at places which have been under our own observa-

TABLE 119.—LOSS IN CAPACITY OF ENGLISH CONTACT BEDS

(Fifth Report, Royal Commission on Sewage Disposal, page 60)

Place	Suspended matter in liquor treated (hourly samples in dry weather, parts per 100,000	Age of beds in months	Av. times beds were filled per day	Million Imp. gal. treated	Loss of capacity, Imp. gal.	Rate of loss of capacity, gal. per million Imp. gal.
Guildford............	15.9	46	2.0	223.8	108,794	486
Exeter (main works)..	12.5	37	1.62	2,056.0	750,750	365
Andover.............	11.1	38	1.2	96.0	79,600	828
Exeter (old works)...	8.2	109	1.64	196.2	39,670	202
Slaithwaite.........	7.1	84	2.0	226.2	33,755	149

tion, where septic tank liquor was treated on primary contact beds of medium-sized material.

"With the exception of Andover, the loss of capacity per 1,000,000 gal. was, approximately, in proportion to the amount of suspended matter put upon the beds. The high rate of loss of capacity at Andover is accounted for by the fact that the material in the beds disintegrated to such an extent as to cause the surface level to sink considerably." (*Ibid.*, page 60.)

English engineers hoped that by the removal of suspended matter the life of contact beds might be made very long. Latham, Frankland and Perkin said (Manchester report, 1899):

"The capacity of bacterial contact beds has been found to remain practically constant after they have been in operation for a period of 3 months."

Clowes and Houston said in 1904:

"The sewage capacity of the coke bed, when the bed is fed with settled sewage, fluctuates slightly, but undergoes no permanent reduction. The bed does not choke, and its purifying power undergoes steady improvement for some time." ("Experimental Bacterial Treatment of London Sewage," page 34.)

Dunbar, however, took exception to this idea from the start. He said in 1907:

"Our results were not in agreement with those of other observers. Even after years of observation, the experts of various English towns stated that only at first was there any appreciable decrease in the capacity of contact beds, and that it soon became practically constant. Our experiments gave a different result, and experience has now shown that we were in the right.

"The experts who formerly opposed our views now exhibit with pride the apparatus which they have in the meantime constructed in order to regenerate their sludged-up contact beds. After about 5 years' operation, the regeneration of the contact beds has proved to be necessary, and is carried out in a manner which, as a result of our experiments, we characterized 10 years ago as the only possible one. Periods of rest and the raking over of the beds do not produce the necessary result. The beds must be taken to pieces and the adhering sludge washed from the material." ("Principles of Sewage Treatment," page 178.)

Fuller said in 1912:

"When the average size of the material is from 1.0 to 1.5 in. in diameter, the present indications are that with good management it is unnecessary to remove and clean the material on account of clogging. This permanency of construction is obtained at a considerable cost, in that it involves the use of false bottoms, final or intermediate settling basins for treating the decomposable solids in the effluent, and perhaps a greater quantity (depth)

of material to afford equal treatment of the sewage, as compared with beds of fine material. The influence of the rate of filtration with beds of fine and coarse material depends upon local conditions.

"As well stated by Dr. Dunbar, it is a financial proposition to determine which is preferable for a given problem: fine material with its expense to remove clogging, or coarse material with less expensive maintenance but greater first cost for installation." ("Sewage Disposal," page 683.)

Washing Material.—The predictions in England regarding permanency of contact beds have not been wholly realized, and it has been found necessary, in many cases, to remove and wash the material after from 3 to 5 years' use.

"The evidence shows that in many cases it is practicable and economical to wash filtering material which has become clogged. The cost varies according to local circumstances, and the following instances may be cited:

"At Burnley, the material of several clogged beds was washed by means of an inclined rotary screen, 10 × 4 ft., fixed at a suitable elevation and so constructed as to be easily capable of removal from bed to bed as required. The arrangement admitted of washing, grading and screening the material (clinker); the washing was effected by the tank effluent, and the power was supplied by a portable engine.

"The cost of these operations was at the rate of 1s. per cubic yard of material, which covered excavating, packing troughs, wheeling, and screening and washing not only the material excavated but also the additional material required to take the place of the old material rejected (about 10 per cent.), and filling it into the bed.

"At Leeds, Mr. Harrison, the chemist to the Corporation, washed 1020 cu. yd. of material by hand labor with screens and a heavy flow of water, at a cost of 2s. 5d. per cubic yard.

"At Manchester, the washing of filtering material has been done on a very large scale, and is carried on continuously. The method employed is to pass the material from a sump to a jigging screen of ¼-in. mesh, over which are fixed a series of horizontal water-sprayers. The material rejected by this screen is further graded by passing over a fixed 2-in. mesh screen. The material which passes through the ¼-in. screen falls on to an inclined fixed screen of ⅛-in. mesh. All material above ¼-in. is replaced in the primary filters; that between ¼-in. and ⅛-in. will be used for the surface of the secondary contact beds, when these are constructed. Settled sewage is the liquid used in the washing.

"The total cost of removing, washing, screening and replacing in beds and making up to original level with new material at Manchester, is 1s. 6d. per cubic yard. This does not include cost of washing machinery." (Fifth Report, Royal Commission on Sewage Disposal, page 63.)

TREATMENT OF DIFFERENT SEWAGES

Septic Sewage.—At Columbus, Johnson found (1905) that there was no advantage, aside from the benefits to be derived from the re-

moval of suspended matter, in the septic-tank treatment of sewage before applying it to contact beds.

Clark and Gage found that any advantage from the application of treated sewage was largely mechanical, that is, the clogging by suspended matter is largely reduced and a considerable load is removed from the bed in that these suspended matters do not have to be taken care of by biological processes. They found it impossible to obtain a satisfactory effluent when strong septic sewage was applied, but when the sewage was first thoroughly aerated satisfactory nitrification followed. (Report Mass. St. Bd. Health, 1908, page 446.)

At the Technology Experiment Station, experiments, although of too limited a nature to furnish conclusive results, indicated that in the case of Boston sewage, septic action of 30 hours' duration was distinctly harmful to further purification in contact beds, although of considerable benefit in the prevention of clogging. It was concluded, however, that a shorter period of septic action, approaching more nearly the conditions of plain sedimentation, would probably maintain the capacity of the beds, without corresponding harmful effects. (U. S. Water Supply and Irrigation Paper No. 185.)

Strained and Settled Sewage.—In June, 1901, 2 contact beds were started at the Lawrence Experiment Station to study the operation under identical conditions of beds receiving untreated sewage and sewage strained through coke or coal. Each bed was 5 ft. deep and constructed of coke, 75 per cent. of which was ½ to 1 in. in size and 25 per cent. from ¼ to ½ in.

From June 3 to October 1, 1901, the beds were flooded four times at intervals of 1 hour, stood full 2 hours and then drained slowly. From October 1 to October 13, 1901, each was flooded four times at half-hour intervals, and stood full 4 hours before being drained. From October 14 to December 15, 1901, the dose was applied in one charge in about 45 minutes and the period of standing full was 2 hours. From December 16, 1901, to February 28, 1902, each bed was filled twice a day in this manner. From March 1, 1902, to the end of 1909, both filters were filled once a day. During the first year the beds were operated without resting, but after June 6, 1906, they rested 1 week in 6.

At the end of this period bed 176, receiving untreated sewage up to January 1, 1907, and settled sewage afterward, was badly clogged with organic matter. The effluent was of poor quality and discolored with iron during most of the year, and attempts to regenerate it by resting and standing full were so unsuccessful that the bed was discontinued. The bed had been dosed during its period of service at the following rates, in gallons per acre daily for 6 days in a week: 1901, 849,400; 1902, 698,700; 1903, 579,800; 1904, 465,000; 1905, 484,700; 1906, 490,600;

1907, 357,500; 1908, 330,100; 1909, 270,000. The deterioration in the quality of the effluent of this bed began in the middle of 1906 and gradually increased even after the change from untreated to settled sewage as the applied liquid. From that time on the effluent was generally putrescible.

Bed 175, receiving strained sewage, was dosed at the following rates in gallons per acre daily for 6 days in the week: 1901, 909,800; 1902, 768,200; 1903, 596,300; 1904, 469,600; 1905, 483,700; 1906, 497,300; 1907, 431,900; 1908, 431,200; 1909, 415,500; 1910, 359,000; 1911, 318,000. At the beginning of 1911, about 51 per cent. of the original open space was filled with deposited matter and during the year the effluent was discolored with iron, the nitrates were very low and the free ammonia was high, indicating a reducing action within the bed. At the close of 1911, the coke was removed, washed and replaced in a new tank.

SIZE OF BEDS

The size of the unit is a matter of more importance in the case of contact installations than in most other methods of treatment.

The unit should be small enough so that under normal conditions of sewage flow the time required for filling shall not be unduly long. Large units are also inadvisable, on account of the long time required for draining. At Manchester, England, with a flow of sewage of about 36,000,000 gal. per day, the individual filters are each one-half acre in area.

At Plainfield, N. J., and Mansfield, Ohio, where the flow of sewage is about 1,000,000 gal. per day, the areas of individual units are 0.22 and 0.25 acre respectively.

DIBDIN SLATE BEDS

Dibdin, who carried out the original experiments with contact beds at London, was impressed with the loss of capacity which such beds undergo and with the great accumulations of sludge resulting when most of the suspended matter is removed from the sewage. Further experiments by him, with the object of applying crude sewage to beds so that the suspended matter could be oxidized by aerobic bacterial action, and in such a way that clogging could be prevented or the beds cleaned without the removal of material, lead to the development of the slate bed. After considerable laboratory work, an experimental installation was built at Devizes in January, 1904. This bed was composed of slates about ¼ in. thick and supported 1 in. apart by means of slate blocks. Under this arrangement the water capacity amounted to 87 per cent. After being in operation for 14 months the capacity was reduced to 50 per cent. It was then roughly flushed out, when the capacity rose to 64 per cent. Some slates were then removed so that

the bed could be flushed with a hose, which resulted in a return to a capacity of 82 per cent. As a result of these experiments the town of Devizes installed a complete system on this basis. Plants have also been built for other towns in England and for private residences and institutions. The working of slate beds was examined at a number of places by the officers of the Royal Commission on Sewage Disposal, from whose report the following notes are taken:

"As the result of our inspection of the beds at Devizes, we came to the tentative conclusion that primary beds containing large slabs of slate must be regarded more as preliminary settling or septic tanks than as contact beds. As regards the effluent which results from the treatment of sewage in slate beds, this conclusion has been borne out by the observations which have since been made.

"It is clear, therefore, that the practical rules which apply to the subsequent treatment of settled sewage and septic-tank liquor apply also to the treatment of the effluent from slate beds.

"The observations at Devizes and Dereham show that single contact of the slate-bed effluents from strong and average sewages cannot be relied upon to produce a satisfactory effluent. The single-contact beds yielded at both these places opalescent and strong smelling liquids.

"At Machynlleth, a good non-putrescible effluent was being obtained by the treatment of a weaker slate bed effluent on percolating filters, at a rate of about 40 gal. per cubic yard per 24 hours in dry weather.

"The capacity of a new slate bed is about 85 to 90 per cent. of the capacity of the empty tank. From the measurements of the capacity of the slate beds at the three places under observation, it is clear that the greater part of this capacity can be maintained permanently if the sludge which collects in the beds is habitually allowed to come away after the beds have been run off.

"At Devizes, after having received for 5 years an average of 0.9 filling per day of sewage containing about 43 parts of suspended matter per 100,000, the slate beds retained 75 per cent. of their original empty-tank capacity. During the period of observation, *i.e.*, in the fourth and fifth years of their life, the capacities of the 2 beds gaged remained practically constant. Here the valves of the beds had been opened every morning to let out the accumulated sludge.

"From the data which have been obtained with regard to Devizes, however, the digestion, or, more correctly, the diminution in solids, which takes place in the slate beds there would appear to be very small. The actual figure obtained was only 1 per cent. of the total suspended solids.

"In this case the production of slate-bed sludge per 1,000,000 gal. was about 1.7 tons, containing 90 per cent. of water.

"Provided that the sewage is not allowed to remain too long in the slate beds, the slate-bed sludge differs from ordinary sewage or septic-tank sludge in that it possesses only a slight odor, resembling that of sea-weed, and that it is full of minute forms of animal life. When examined under the micro-

scope, the number of actively motile vibrios is a striking feature. Numbers of small worms are also to be found in the sludge as it lies on the slates.

"If the sludge after removal is allowed to remain wet, it putrefies and gives rise to an offensive smell, but there is little doubt that when drained quickly in a shallow layer it can be dealt with almost without nuisance. It is to be clearly understood, however, that if this satisfactory result is to be attained, a sufficient number of thoroughly good draining beds or lagoons must be provided for the reception of the sludge; when drained, it can be dealt with at leisure.

"The smell which arises from sewage works where slate beds are installed may be taken as similar in kind and intensity to that which occurs where crude sewage is treated in contact beds. Once the sewage is in either type of bed, there is practically no smell, but during the filling of the beds solid sewage matter collects on the surface of the material, whether slate or clinker, etc., and necessarily gives rise to local nuisance. These deposited solids can, however, be swept into the slate bed as it fills up.

"As regards the slate-bed effluent itself, it may be said that this is more liable to give rise to nuisance, when distributed, than precipitation liquor, but less liable than septic-tank liquor.

"Compared with other preliminary processes of sewage purification, slate beds will probably be found to be expensive as regards capital outlay; but, on account of the sludge being comparatively odorless, we think that the claims of the process deserve consideration in cases where the reduction of smell at a sewage installation is of primary importance." (Seventh Report, Appendix III, page 202.)

CONTACT BEDS IN UNITED STATES

Contact beds have not been extensively used in the United States, principally because of the rapid development of, and lower cost per unit of work accomplished by, trickling filters. Between 25 and 30 installations have been made, however, although most of them are quite small. In many cases this type of bed is more adaptable for small works, such as institutions and private residences, than the trickling filter. It requires less head than trickling filters, which is of considerable importance at times. It is practically inodorous, the sewage is out of sight and the plant may be operated with a fair degree of reliability by automatic devices. It can also be used to greater advantage than the trickling filter, under severe winter conditions. In fact, at Columbus, Ohio, the trickling filters have been piped in such a way that they may be operated as contact beds whenever a long period of cold weather may make this desirable, but up to the present time this has not been found necessary.

One other condition which might under some circumstances lead to the use of contact beds may be mentioned, that in which denitrification of the effluent is desirable. Green algæ feed upon nitrogen in the form of free ammonia or nitrates, and in some cases where ex-

cessive growths have occurred they have been attributed to the presence of sewage or sewage effluent. At Belfast, Ireland, where very troublesome growths of sea lettuce (ulva latissima) have occurred they have been attributed to the pollution of the bay by the Belfast sewage. As there is a considerable loss of nitrogen from some contact beds, Letts proposed (1908) that the sewage of Belfast, after passing through trickling filters, should be treated on contact beds for denitrification, and showed by experiments that this loss of nitrogen would be very large. The Royal Commission on Sewage Disposal, however, said in its Seventh Report:

"The novel scheme of denitrification proposed by Professor Letts has many features of value and interest. It was considered at that time of great importance to eliminate as far as practicable from Belfast sewage the substances which especially nourished the growth of the weed. In such circumstances, the scheme elaborated by Professor Letts for the removal of nitrogen from the sewage would merit most careful consideration. But we are now satisfied that the nuisance cannot be effectively removed by any scheme limited to the treatment of Belfast sewage. Assuming that Belfast sewage has in the past greatly encouraged the growth of ulva and assuming also that this source of nourishment were suddenly cut off, we think that the disappearance of ulva would by no means necessarily follow. Other sources of chemical nourishment of the weed would remain available in the Lough—temporarily if due to deposit on the banks from past sewage discharges, more permanently if resulting from the decay year by year of ulva fronds, the supply of which would be seasonally renewed; moreover, it must be remembered that even if a complete scheme of filtration were adopted, a large amount of unfiltered sewage would, in times of heavy storms, necessarily find its way into the Lough. We do not, however, consider, for the reasons which have been indicated, that pollution is essential to the growth of ulva. The removal of sewage pollution could not, therefore, be a complete and certain remedy for the evil" (page 11).

Plainfield, N. J.—The largest American plant, and the only one where continuous and scientific studies of operation have been made, is at Plainfield, N. J. At this place 8 primary and 8 secondary beds have been built, one-half in 1901 and one-half in 1905, with a total area of about 3.6 acres. The average flow of sewage is about 1.7 million gal. per day. Scarcely any trade wastes reach the sewers.

The sewage is first passed through 4 septic tanks, described in Chapter XI, holding about 14 hours' flow. During 1910 the primary beds were filled from below. The general method of operation has been to drain each bed immediately after filling because of the lack of capacity and the poor draining of the partially clogged beds, as stated by R. S. Lanphear, *Engineering Record*, Vol. lxiv, page 29.

Hourly determinations of dissolved atmospheric oxygen in the sewage show that the quantity varies from less than 1 part per 1,000,000, during

the afternoon hours, to from 6 to 7 parts in the early morning hours. The effluent from the primary beds contains practically no dissolved oxygen while that from the secondary beds contains a substantial quantity—the lowest between May, 1910 and May, 1911, being 2.22 parts per 1,000,000 in June and the highest being 3.69 parts in February.

The putrescibility of the effluent was tested by allowing samples to remain in closed bottles at room temperature for 48 hours, testing them at the end of that period for the presence of dissolved atmospheric oxygen. If oxygen was found to be present, the sample was reported as non-putrescible; if oxygen were absent, the sample was said to be putrefactive. The primary-bed effluent even when diluted 1:1 with tap water was found to be putrescible by this test, which does not appear to be as severe as the methylene-blue test. Samples of the secondary effluent when tested undiluted were reported non-putrefactive from January to May inclusive; putrefactive from June to September inclusive, and non-putrefactive from October to December, inclusive. When diluted 1:1 with tap water, the tests indicated non-putrefactive effluents throughout the year.

TABLE 120.—MONTHLY SUMMARY OF RESULTS OF ANALYSES OF SEWAGE AND EFFLUENTS, PLAINFIELD SEWAGE TREATMENT WORKS, 1910

1910	Quantity of sewage, mil. gal.	Parts per million												Bacteria					
		Suspended matter				Oxygen consumed				Nitrogen as nitrates		Nitrogen as nitrites		Millions per cubic centimeter				Per cent. removed	
		Screened sewage	Septic effl.	Primary effl.	Secondary effl.	Screened sewage	Septic effl.	Primary effl.	Secondary effl.	Primary effl.	Secondary effl.	Primary effl.	Secondary effl.	Screened sewage	Septic effl.	Primary effl.	Secondary effl.	Filters alone	Entire system
January...	1.9	116	50	18	7	74	56	27	10	0.9	3.5	0.16	0.08	....	....	....	....	...	...
February..	1.85	114	50	20	12	74	53	25	14	0.9	2.3	0.12	0.04	1.27	0.95	0.51	0.19	80	85
March.....	1.7	133	58	19	9	78	56	26	11	0.4	2.9	0.13	0.08	2.04	2.44	1.09	0.52	79	74
April......	1.7	138	45	24	12	71	50	27	12	0.5	4.4	0.10	0.20	2.27	1.43	0.79	0.38	73	83
May......	1.65	168	42	19	7	74	52	27	11	0.2	3.1	0.03	0.17	2.66	1.44	0.82	0.47	67	81
June......	1.7	156	47	32	10	72	51	34	12	0.1	2.9	0.01	0.26	2.87	1.40	0.77	0.44	69	81
July.......	1.7	142	65	32	16	62	49	29	12	0.1	2.2	0.01	0.27	2.67	1.26	0.78	0.44	65	83
August....	1.65	146	55	25	7	60	46	26	8	0.3	3.1	0.01	0.16	2.96	1.69	1.09	0.71	59	76
September.	1.7	152	55	38	12	72	50	29	9	0.2	2.9	0.01	0.15	2.59	1.43	0.81	0.46	68	82
October....	1.5	196	60	31	11	86	55	31	11	0.3	3.4	0.04	0.14	2.22	1.09	0.56	0.33	70	85
November.	1.6	271	69	32	11	100	60	29	10	0.8	4.4	0.14	0.15	2.37	1.57	0.97	0.51	67	78
December..	1.9	191	72	29	10	92	60	29	10	0.5	4.2	0.07	0.09	2.09	1.25	0.66	0.28	78	87
Average...	1.7	152	56	27	10	76	52	28	11	0.4	3.3	0.07	0.15	2.36	1.45	0.80	0.43	70	82

(Ref. "*Eng. Rec.*" Vol. 64, page 29.)

The results of chemical and bacterial analyses of the screened sewage and septic tank, primary contact bed and secondary contact bed effluents, from January to December, 1910, are given in Table 120.

Lanphear makes the following observations in *Engineering Record*, Aug. 10, 1912, as to important factors of design and operation:

Contact Material.—"The original specifications in 1900 called for trap-rock varying in size from ¼ to 1½ in. in the primary filters, and in the secondary filters, slag in some beds and cinders in others, somewhat smaller in size than the stone in the upper beds. In the 1905 enlargement, cinders ½ to 1½ in. in size were used in 3 primary beds and 3 secondary beds, and the material in the existing primary beds was washed. The Plainfield plant has yielded a good effluent throughout the 12 years of existence, but the primary beds especially have become greatly clogged. It is believed that trap-rock 1 to 2 in., preferably 1½ in., is more satisfactory than any other local filling material for primary filters. The present material is too fine, and the use of fine cinders (½ to 1 in.) in primary filters is condemned.

"Four-foot depth of filtering material has given satisfactory results. The secondary filters, four of which have been in use 12 years, are considerably clogged, but still hold about one-half to two-thirds of their original capacity and the contents of about 2½ primary filters as they are at the present time. . . .

Underdrainage.—"The underdrainage of each filter consisted of 14 lines of 5-in. horse-shoe tile laid radiating from the gate-chambers, located in the center of each set of 4 filters, and coarse stone was spread between and over them to a depth of 6 in. The floor of each filter had a slope of 1 per cent. toward the gate-chamber. Opening the filters in places in 1909–1910 showed that the underdrainage was inadequate. The beds had evidently unloaded in earlier years and lack of underdrains resulted in the lower portion of the material becoming a wet and black mass almost like mud in consistency. A false bottom affording freer passage of clogging material would result in a longer life for the filter. A greater slope than 1 to 100 would have helped out in places where the floor has settled leaving areas which are always covered with water. It is believed that a drain located at one side of each bed, or on the diagonal, would, perhaps, tend better to distribute the clogging that might be due to lateral motion of suspended solids. At the present time such clogging matter all tends to work toward one corner.

Carrier.—"The 4 outlet gates in the lower part of each upper gate-house deliver the effluent to 1 pipe running to each lower gate-house, which pipe is on about the same level as the outlet gates. This has been found to be a great disadvantage since the primary filters became badly clogged and drained very slowly. Contact filters are generally used where little head is available, and the use of a well in either the gate-house or on the carrier is prohibited. Separate drains from each bed to such a well would be of advantage throughout the entire existence of the plant.

Inlet to Filters.—"Until 1909 all filters were filled from the top, the

distributors being laid, in the earlier years, in 12 in. of coarse stone on top of the working material. Such an arrangement was abandoned and the top stone removed on account of clogging below. This made the removal of scum a much easier matter. In 1909 the primary filters, although clogged and inadequately underdrained, were changed so as to fill from below in order to avoid odor from the exposure of the septic-tank effluent. This was done in connection with some temporary work and worked successfully for a short time. Now the water appears on the surface of the primary filters almost immediately upon commencing to fill.

"The results show that filling from below reduced the amount of nuisance, also that coarser material and better underdrainage are absolutely necessary. It is believed that with 1½-in. material and a false bottom, good results can be obtained from a primary filter, although it may be necessary to settle the effluent in order to remove suspended matter. The absence of scum means a great deal toward the good appearance of the plant, as scum from septic effluent especially seems to be most excellent fertilizer for weeds. There would also be some hesitation toward using material less than 1 in. in size in the secondary filters, if they were to be filled from below.

Vents to Underdrains.—"In certain primary beds a number of 8-in. pipe are set up as sight pipes to the underdrains. If the slope of the floor of the filter is slight and the filters are filled from below, our experience shows that these vents should be made tight and continue up above the material, perhaps as much as 18 in. Any direct strata of less clogged material means that the influent to the filters rises in this pipe, bringing solids from the underdrains and depositing them upon the surface of the filter.

Dosing Devices.—"No automatic dosing devices have ever been used at Plainfield. The installation of such devices at a cost of about $5000 for this plant, with a capacity of 3,000,000 gal. daily, would obviate the expense of a night attendant whose wages are $600 per year. The day man would have about 4 to 5 hours' additional each day in which to attend to scum, weeds and such work.

Efficiency of Plant.—"The Plainfield plant in its present condition removes about 85 to 90 per cent. of the suspended solids (20 to 25 per cent. by filters) and 80 to 85 per cent. of the organic matter as determined by oxygen consumed (about 50 per cent. by filters). The bacterial efficiency is from 65 to 80 per cent., some 80 to 90 per cent. of which is the work of the filters.

"In view of Plainfield experience considerably longer life for a plant with possibly slightly less efficiency should be sought, even though the initial expenditure may be slightly greater."

ADVISABILITY OF AUTOMATIC CONTROL

In considering the advisability of automatic control, it must be remembered that both the quantity and the quality of sewage vary greatly from hour to hour and from day to day, and that the capacity and efficiency of the beds also change from time to time. Night sewage is much weaker than that received during the middle of the day. In a

small plant automatically controlled, where there are only from 3 to 5 beds, it is possible that the same bed may receive day after day the strong sewage of the day time, while the other beds, receiving the weaker sewage of the night, early morning and evening, are called upon to do less work. In most cases, however, the fluctuations in flow from day to day will cause more or less change in the hourly cycle, and under such conditions the use of automatic dosing apparatus will probably insure better work on the part of the beds and less expense for caretakers than the operation of the controlling gates by manual labor. In large plants, however, the complication of the apparatus is so great and the beds vary so much in their capabilities that it is probable that better results can be obtained by the intelligent operation of gates by manual labor than by automatic dosing apparatus.

The Royal Commission on Sewage Disposal said in its Fifth Report:

"We have had many opportunities of studying the use of automatic apparatus for filling and emptying contact beds, and we have received evidence from several witnesses who have had considerable experience of these devices.

"Our own observations and the experience of others show that it is not possible to rely entirely on automatic apparatus for sewage works, although within certain limits it may be advantageously used with considerable saving of labor.

"In the case of large sewage disposal works, where men are always available, we consider that it would usually be inexpedient to provide automatic plant. It is liable to get out of order, it does not adapt itself to variations in volume and strength of sewage, state of beds, etc., and generally it is preferable to work the beds by manual labor, where such labor is at hand.

"At Manchester, automatic gear was given a careful trial, with a view to its ultimate adoption if successful. It had, however, to be abandoned, and in reference to this Dr. G. J. Fowler stated: 'I have had considerable experience of these devices, both on the experimental and working scale at the Davyhulme works and also at the Withington works. My experience leads me to think that in large works, where skilled supervision must be provided in any case, there is no advantage in their use. In small works I think some simple mechanism is useful, chiefly on economic grounds, but it should not be left without some attention, if possible, at least once a day. In small works, with the filtering area divided into few units, great fluctuations may occur in the cycle unless some arrangement of dosing tank is provided, or storage in the main tank (septic or other). These fluctuations may be harmful by causing the time of filling to be too prolonged, with consequent curtailment of the resting period. The chief difficulty, however, is the constant variation in the water-holding capacity of the beds and in the permeability of the surface, causing pondage and irregular working' " (page 55).

Fuller, however, who has had considerable experience with the Plainfield, N. J., works, said in his "Sewage Disposal:"

"The author is strongly in favor of using some style of the several admirable arrangements available in America for controlling automatically the operation of contact beds. At Plainfield, N. J., it is recommended that this be done in the interests both of efficiency and economy. For an investment of about $5000 a plant of a capacity of 3,000,000 gal. daily may obviate the expense of a night attendant. This applies of course to filters of moderately coarse material where it is unnecessary to remove scum and deposits associated with clogging, and where advantageous use is taken of preliminary and intermediate settling basins" (page 864).

COST OF OPERATION

Probably the best figures on the cost of operation in America are those obtained at Plainfield, N. J., stated by Fuller to be as given in Table 121.

TABLE 121.—COST OF OPERATION OF PLAINFIELD SEWAGE DISPOSAL PLANT

Item	1907	1908	1909	1910
Manager-chemist, consulting engineers.	....	$1325.50	$1818.46	$1677.67
Night operator	$540.00	....	....	....
Laboratory	41.69	247.87	147.18	80.72
Tools and supplies	23.02	103.45	32.63	8.28
Labor	50.59	53.70	....	....
Water guarantee	73.20	73.20	73.20	....
Telephone	43.99	25.08	28.58	23.05
Care of contact beds	1180.53	1189.26	885.09	918.68
Care of septic tanks, including emptying and disposal of sludge.	662.25	603.50	252.89	269.17
Grading and weeding banks	104.22	....	....	....
Screen attendance	....	193.14	298.30	312.23
Farming	236.15	....	....	....
Total	$2955.64	....	....	....
Farm products receipts	248.65	....	....	....
Total cost of maintenance	$2706.99	$3814.70	$3536.33	$3289.80
Improvement of contact beds	....	....	2032.87	935.36
Repair of septic tanks	101.14	....	151.89	1011.15

In 1910 the number of connections was 3746; assuming 5 persons per connection, there would be a total of 18,730 persons. The flow

amounted to 1,800,000 gal. per day, making the cost $5 per 1,000,000 gal. or $0.18 per capita per year.

At Mansfield, Ohio (Report Ohio State Board of Health, 1908), the costs of operation during 1906 and 1907 were $5644 and $5260 respectively, and included removal of sludge from the septic tanks. Furthermore, about one-half of the cost was for coal used in pumping. These figures make the per capita cost $0.47 and $0.44 respectively.

At Manchester, England, very complete cost accounts have been kept. In the 1907 report of the Rivers Department is given a table showing the actual cost of a selected area of 6 acres from the starting of the beds until the filtering material was taken out:

Average number fillings..........	2,690
Gallons (U. S.) of septic tank effluent dealt with by the 6 acres...	4,610,000,000
Total maintenance cost..........	$4,085
Total renewal cost ($0.40½ per cubic yard)..................	$13,700
Maintenance cost................	$1.05 per 1,000,000 U. S. gal. filtered.
Renewal cost....................	$3.57 per 1,000,000 U. S. gal. filtered.
Actuating valves................	$0.30 per 1,000,000 U. S. gal. filtered.
Total working cost..............	$4.92 per 1,000,000 U. S. gal. filtered.

CONSTRUCTION

The Royal Commission on Sewage Disposal said in its Fifth Report:

"In some cases contact beds have been made by simple excavation, but our experience and the evidence which we have received show that in the majority of cases it is desirable that the beds should be constructed of building materials.

"When contact beds are constructed by simple excavation in clay soil, there is risk of the clay sooner or later, according to the character of the stratum, working up into the filtering material. Moreover, the lines of underdrains are apt to become distorted, as the result of unequal settlement.

"At Heywood an experimental bed was constructed, without concrete, in clay soil, and it was found that the clay worked up into the bed and formed an impervious mass round the drain pipes.

"Again, there is danger of leakage unless the soil is of a very dense nature and the embankments are of considerable width. This leakage may occur as a result of unequal settlement, or through rats or moles burrowing into the embankment, or through the cracking of the clay during periods of rest or frost. The result of leakage may be detrimental to the working of the bed. Two instances which have come under our own observation may be mentioned:

"At Halton it was found necessary to fill two of the primary contact beds simultaneously, as the liquid percolated from one bed to the other through the intermediate embankment.

"At Oswestry, where double-contact beds were constructed in fairly stiff clay soil, some of the retaining banks became sodden with liquid after 3 or 4 years' work and sank considerably in consequence.

"Even in the most impervious kind of soil the outlet chambers of contact beds must be constructed of building materials, and, unless this work is most carefully carried out, there is danger of leakage at the juncture of the chamber and the embankment. It may also be remarked that the embankments required, where no building materials are used, take up more space than properly built walls.

"On the other hand, where the site for the works consists of a dense clay, experience shows that the construction of contact beds by simple excavation may be satisfactory. As examples, we may refer to the beds at Burnley and Oldham" (page 51).

In the United States concrete is generally used for walls and bottom, although at Mansfield, Ohio, the beds were constructed with earthen embankments.

An unusual form of contact bed has been developed by Alvord. It was first used at Bloomington, Ind., in 1909, at a plant designed to treat 500,000 gal. of medium domestic sewage with a small quantity of trade wastes. The sewage is not over 2 hours old on reaching the works. The latter comprise an octagonal settling tank, surrounded by an octagonal group of contact beds which discharge their effluent upon sprinkling filters. Around the outer boundary of these filters is a ditch serving as a secondary sedimentation basin.

The sewage has about 11 hours' detention in the settling tank, which is divided into 5 compartments. The settled sewage passes from these compartments over weirs into the 4 "combined-contact dosing chambers." Each chamber has a false floor of stone slabs carried by pillars, leaving a clear space 3 ft. high below the slabs. The latter support a bed of 1½ to 3-in. crushed stone. In a corner of each bed is a triangular dosing chamber containing an 8-in. siphon which discharges intermittently the sewage in the top 5½ ft. of the bed over its corresponding trickling filter. If the voids in the broken stone are 33 per cent. of the volume of the contact bed, the dose at each discharge of a siphon is about 7000 gal. The chamber below the bed is constantly full of effluent, which Alvord believes to be somewhat changed at each discharge of a siphon; the sediment in the chamber is flushed out at intervals through the sludge pipe of the central settling tank. The rapid siphoning of the liquid contents of such a bed washes solids from the stones into the chamber below; after 4 years of operation under very high rates the beds were free from clogging. There has been no trouble with nozzle clogging on the trickling filters, according to Anton Littler, superintendent of the plant.

The rates at which these beds operate are so high as to render it a question whether they should be classed as contact beds or strainers.

The 4 beds have a capacity of about 28,000 gal., but ordinarily half the plant is shut down for recuperation, so that the average daily flow of 800,000 gal. is handled by 2 beds of only 14,000 gal. capacity, making it necessary to discharge each bed fifty-seven times daily.

This type of bed is used in the sewage treatment works designed by Alvord for Madison, Wis.

The largest and in many respects the most interesting installation of contact beds thus far built is at Manchester, England. A special report upon this plant was prepared by Fowler and Wilkinson, in 1902, from which the following description and illustrations have been taken.

In 1902 the population served by the 1700 miles of sewers within the city was 564,000. The water-supply at that time was estimated to be 35 U. S. gal. per capita per day, of which 19 gal. were for domestic purposes, the remainder being used by the industries. It is surprising to American engineers to find that at this date there were in use, in Manchester, 73,915 pail closets and only 45,686 water-closets. The sewer system receives storm water, including which the average daily flow of sewage in 1901 was over 40,800,000 U. S. gal., the average daily dry weather flow being 30,800,000 gal.

The project for contact beds, adopted by the Manchester City Council in 1900, involved the construction of 46 acres (net) of first-contact beds, each ½ acre in area; 26 acres of storm-water filters, each about 1 acre in area and 2 ft. 6 in. deep; and 46 acres (net) of second-contact beds, subsequently reduced when experience showed that the area originally proposed was not needed. This project was designed to deal with a maximum rate of flow of 151,200,000 U. S. gal. in 24 hours in time of storm, the flow up to half that amount being treated by double contact on the contact beds and the other half on the 26 acres of storm-water filters. The excessive sewage flow in time of rain beyond this maximum rate was to be discharged directly into the Manchester Ship Canal.

Fig. 127 shows the general arrangement of the underdrainage and distribution systems of a typical bed. It will be noticed that the bottom of the bed is graded in ridges and furrows to facilitate drainage toward the underdrains. Both the distribution and underdrainage systems radiate from the center of the side wall, parallel to which a single channel is provided which serves both beds. The sewage from this channel flows into a small distributing reservoir, from which it passes over a circular weir and thence along channels formed in the surface of the bed, as illustrated by Fig. 128. These channels are lined with fine material which tends to arrest suspended matters and prevent them from entering the bed. It will be seen from Fig. 127 that the underdrainage system radiates toward a collecting channel, which is con-

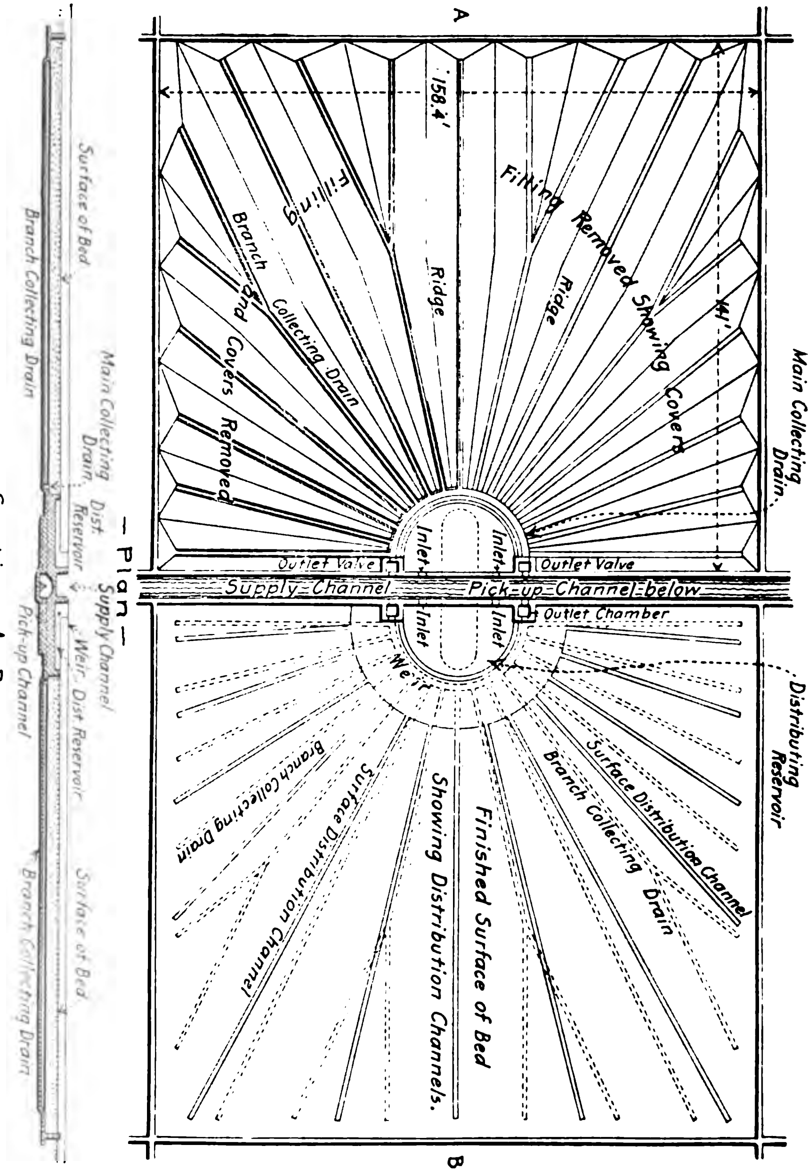


FIG. 127.—Plan of two half-acre contact beds, Manchester, England.

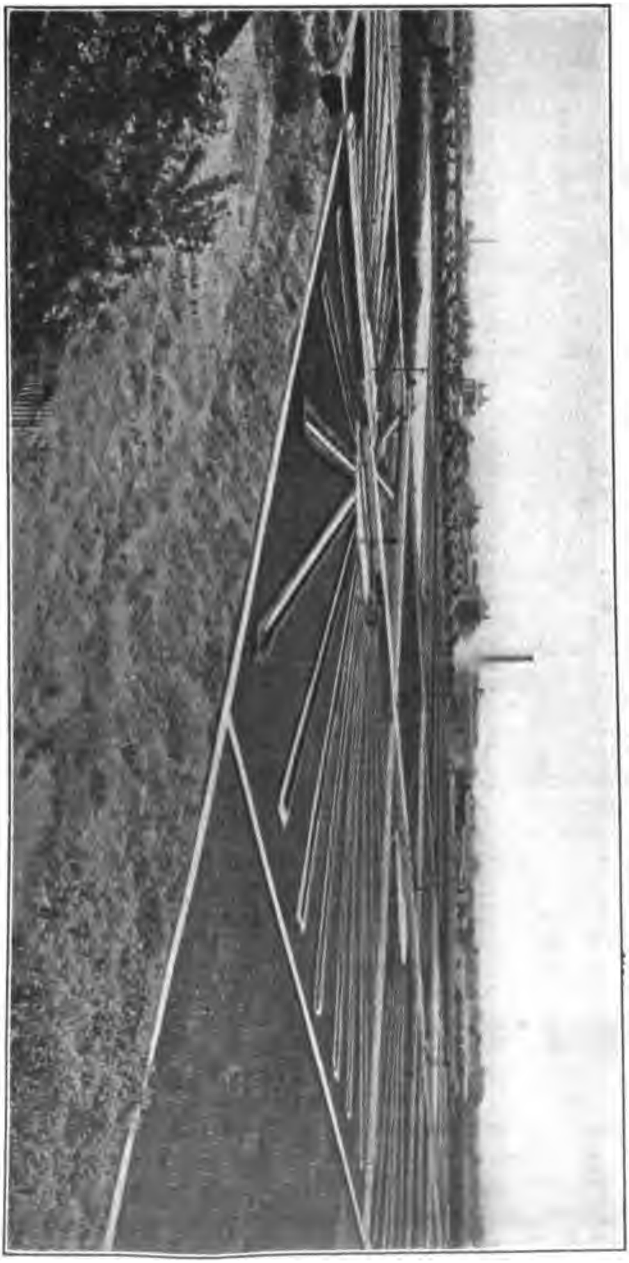


FIG. 128.—Arrangement of distribution channels on Manchester contact beds.

centric with the distributing weir and connects with the main effluent drain. The bottoms of the beds are of concrete, in which the underdrain channels are formed. These channels are covered with perforated stoneware slabs so placed as to be flush with the surface of the bottom.

The details of the distribution and underdrainage systems of a typical storm-water bed are shown in Fig. 129. The floor of the beds is the natural ground surface, generally a clayey marl, covered with a layer of concrete where the character of the ground renders it necessary. The lateral underdrains are of brick laid with open joints on concrete foundations and covered with stoneware or fire-clay slabs. The general scheme of underdrainage is shown by Fig. 130.

The cost of the filters, according to the Royal Commission on Sewage Disposal, Fifth Report, was from $12,500 to $15,000 per acre.

At Mansfield, Ohio, the beds, designed by F. A. Barbour, are laid out in the form of a circle, each of the five 0.25-acre beds forming a sector separated from the next by an earthen embankment. They were described as follows in the report of the Ohio State Board of Health for 1908:

"The filtering material is chiefly crushed cinders, which range in size from ¼ to ¾ in. The cinders were obtained from the Pennsylvania railroad and were screened and crushed at the plant by a special apparatus. They are said to have cost 85 cts. per cubic yard, in place. The total depth of filtering material is 4 ft. 9 in.

"Each filter is underdrained[1] with branched lines of vitrified tile, which comprise 171 ft. of 4-in. tile, 169 ft. of 6-in. tile, 170 ft. of 8-in. tile, 16 ft. of 10-in. tile and 8 ft. of 12-in. tile. The drains, which form a network over the entire bottom of each unit, are laid in depressions 6 in. deep and about 20 ft. apart. They are placed with open joints and are surrounded to a depth of from 4 to 9 in. with coarse cinders which range in size from ½ to 1½ in. The general slope of the underdrainage system is 1.42 ft. in 100 ft. At the end of each underdrain branch, for purposes of ventilation, there is a 4-in. iron pipe which extends several inches above the surface of the filtering material. There are seven of these ventilating pipes in each filter. The underdrains are so laid that the filter is ordinarily drained completely. In case it is deemed necessary to retain a certain quantity of sewage in the underdrains, arrangements were made when the plant was built to close the regular outlet by means of a sluice gate and to cause the effluent to discharge through a second pipe located at a higher level. It appears that this arrangement has never been used.

"Sewage flows from the dosing apparatus to each filter through about 15 ft. of 18-in. iron pipe, which terminates in a concrete culvert at the apex of each filter. The distribution on the surface of each filter is a wooden

[1] The underdrainage is actually much better than is indicated by these figures, for during the laying of the tile numerous 4-in. branches were added which were not called for by the plans.

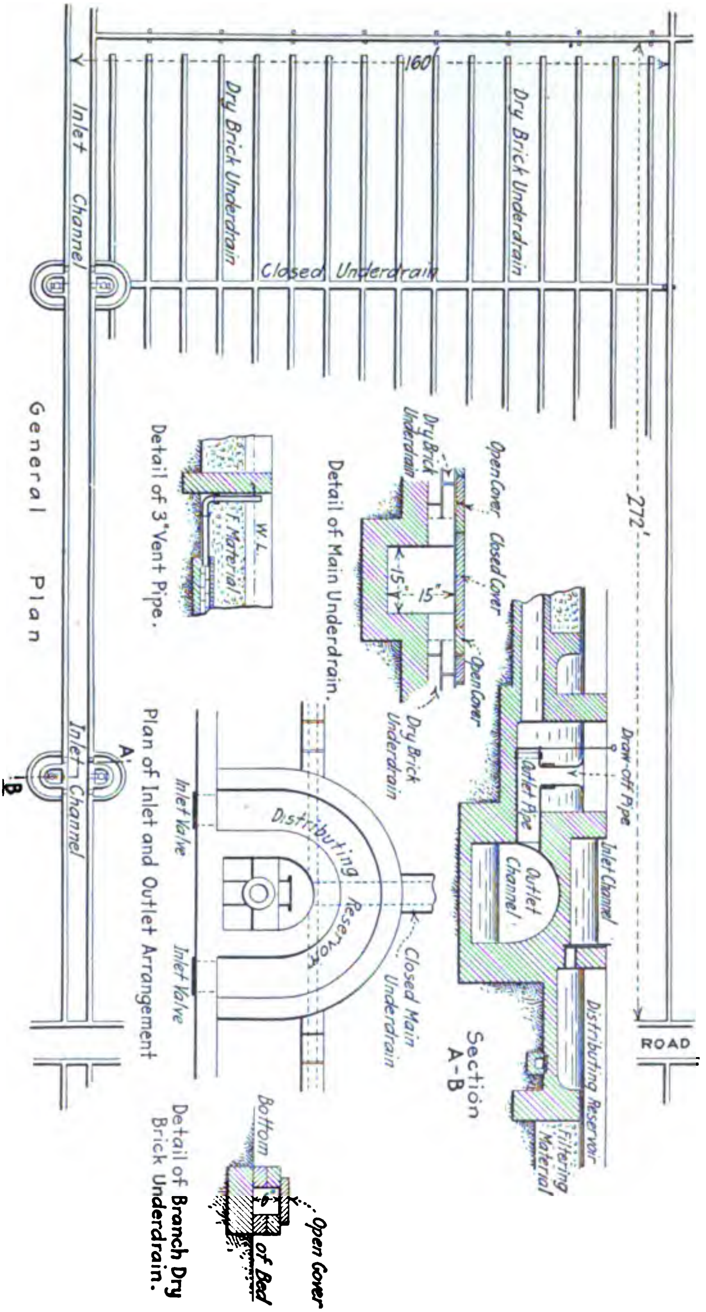


FIG. 129.—General arrangement and details of storm-water beds, Manchester, England.

carrier which ranges in width from 3 ft. 4 in. to 6 in. The distributor is 6 in. deep throughout its length. The dimensions of the distributor in detail are as follows: 15 ft., 3 ft. 4 in. wide; 30 ft., 2 ft. 10 in. wide; 30 ft., 22 in. wide, and 30 ft., 6 in. wide. In addition to this main line, there are four 30-ft. branches, two of which are 12 in. wide and two 6 in. wide. Hinged gates are provided at each change of section to control the aliquot part of the flow discharging at any given point. At each change of section wooden slabs are provided 2 ft. 6 in. × 4 ft. in area."

Control is by the Barbour apparatus described in Chapter XVIII. The contact filters and appurtenances cost about $16,400 per acre.

The contact beds at Mansfield have a bottom of earth into which cinders were rolled to secure as great density as possible. Tests made in October, 1914, showed that the total leakage from the beds was about 5 per cent., mostly through a revolving valve controlling the discharge from the underdrains. The liquid that escaped in this way passed through the contact material and its escape was not considered of any sanitary significance. The beds were found to be free from mud in the bottom.

The plant went into service in 1902 with weak sewage, the dry-weather flow amounting to 765,000 gal. per day. In October, 1914, the dry-weather flow was about 1,899,000 gal. and the sewage had become of normal strength. The sewerage system has a high-level service discharging directly to the plant and a low-level service from which the sewage is pumped to the plant. The records are so imperfect that there is no means of knowing what proportion of the low-level sewage actually reached the treatment works.

During the first 4 years the air space of the beds decreased from 42 to 28 per cent., but it subsequently increased slightly in spite of the additional load. The effluent from the septic tank was not delivered as intended by the designer after the first few years of operation, because lack of attention to the automatic effluent weirs of the tanks, installed to smooth out the hourly variations in flow, resulted in their failure to operate. The filling period was found to vary from 3 hours 28 minutes during the hours of maximum flow to 8 hours 42 minutes at night.

Objection was raised to the plant by lower riparian owners which led to an investigation of its condition by Barbour in 1914–15. He found that the objectionable conditions were due to the discharge of raw sewage into the river at times when the works were unable to treat it, because their capacity was far outgrown, as much as 2,000,000 gal. sometimes being treated on 1¼ acres. He recommended restricting their service to sewage from the high-level system, working the beds on a rate of 600,000 gal., and constructing Imhoff tanks and trickling filters worked at a 2,000,000-gal. rate to treat the sewage pumped from

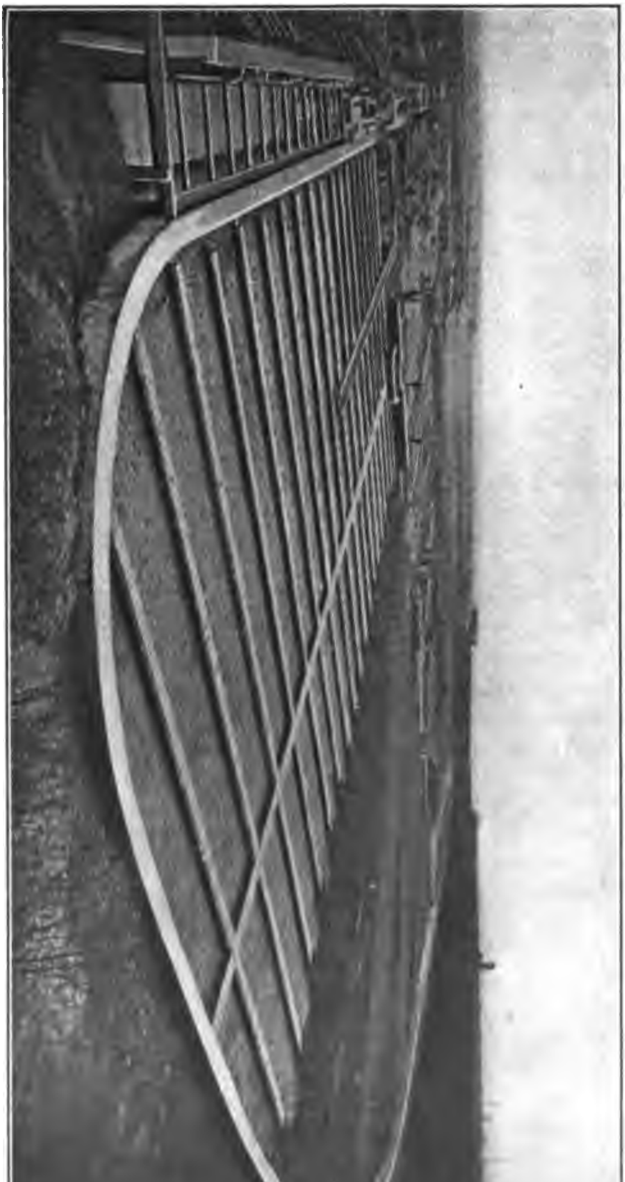


Fig. 130.—Underdrains and filling of a partly constructed storm-water bed, Manchester, England.

the low-level district. This advice was based on the lower cost of the recommended system in comparison with an increase in the area of contact beds, at the rates of operation stated. The operating results of the original plant, when worked at rates not exceeding those for which it was designed, were regarded as entirely satisfactory.

CHAPTER XV

TRICKLING FILTERS

The trickling filter is a bed, usually from 5 to 10 ft. deep, of coarse material such as clinker, gravel or broken stone, through which sewage trickles in thin films. Such beds are known as "trickling," "percolating," "aerating," "sprinkling" or "sprinkler" filters, the last two terms being used more commonly in the United States than elsewhere, probably because the sewage is usually sprayed or sprinkled by pressure nozzles over the surface of the beds in this country; this is shown in Fig. 131, a view of a filter built at Chambersburg, Pa., from the plans of Albright & Mebus.

Character of Filtering Medium.—Theoretically, the greater the surface afforded, the more efficient the filtering medium will be, and experiments have proved the theory to be true. At the Philadelphia sewage testing station, settled sewage was applied to five 18-in. beds, consisting of common marbles, gravel, broken stone, broken slag and broken coke respectively, at the rate of 500,000 gal. per acre daily.[1] The effluents differed greatly, the quality improving in the order of enumeration of the filtering media, the bed of marbles furnishing the poorest and that of broken coke the best. (Report Sewage Testing Station, page 120.) At York, England, 7½-ft. beds of 1½ to 3½-in. broken brick, slag, coke and clinker produced effluents of different qualities, the character improving in the order in which the media are stated. (Fifth Report, Royal Commission on Sewage Disposal, Appendix III, page 514.) It was also observed that the rate of passage of the sewage through the different beds varied inversely as the quality of the effluent, the broken brick furnishing the most free passage. Experiments made at the Lawrence Experiment Station are summarized in Table 122.

Rough materials have the disadvantage that they tend to retard the unloading of transformed solid matters and are, therefore, more liable to become clogged than smoother materials. Clogging interferes with the natural passage of air into the filter and an abundant air supply is of the utmost importance. It may mean that a portion or all of the filtering material must be removed and washed, which adds greatly to the cost of operation. Under ordinary conditions, therefore, it will be well to avoid extremely rough materials like slag as well as very smooth materials like gravel.

[1] All particles were as nearly the same size as possible, but with increasing surface area. The roughness of the materials was also considered an important factor.

Another important practical matter is the permanence of the filtering medium. Certain kinds of materials disintegrate, so that the bed as a whole may in time come to be entirely different in character. At Philadelphia it was found that trap rock and gravel maintained their initial size, while limestone and slag disintegrated to some extent. In some climates the upper portion of the bed is liable to be alternately frozen and thawed during the winter, and if the material is porous there is a decided tendency toward disintegration. Trap rock is a satisfactory material in this respect. Coke and cinders are undesirable. It is conceivable, however, that there may be places where it will be more economical to use cheap, possibly waste, materials like cinders even though they may have to be occasionally renewed, than to build of more permanent but considerably more expensive material like broken stone.

TABLE 122.—TIME OF PASSAGE OF SEWAGE THROUGH EXPERIMENTAL TRICKLING FILTERS, LAWRENCE, MASS.

(Report of the Massachusetts State Board of Health, 1908, page 386)

Material	Size of material, inches	Depth		Ave. daily rate[1]		Time of passage of sewage, hours	Nitrates, parts per million
		ft.	in.	Gal. per acre	Gal. per cu. yd.		
Broken stone.....	1/4-1	10	0	1,150,000	71.3	3	29.5
Broken stone.....	1/4-1	10	0	1,938,000	120.1	2	20.5
Broken stone.....	1/4-1	8	0	963,400	74.6	2	21.2
Broken stone.....	1/4-1	5	0	925,500	114.7	2	5.2
Coarse clinker....	3/4-1 3/4	5	9	885,000	95.4	6	12.4
Coarse clinker....	3/4-1 3/4	3	10	949,000	153.5	1	7.3
Fine clinker......	1/4- 3/4	5	9	896,500	96.6	14	22.0
Fine clinker......	1/4- 3/4	3	10	908,900	147.1	6	5.0

[1] Six days a week.

Size of Filtering Medium.—The smaller the filtering material the greater will be its bacterial surface and the longer will be the time afforded the sewage for contact and oxidation. There is some difference of opinion as to the importance of the length of time of contact. Dr. W. P. Dunbar states in his "Sewage Treatment" (page 140) that if a quantity of sewage equal to the water-retaining capacity[1] of a filter 3 ft. deep is applied to such a bed, 20 to 30 minutes will elapse before that sewage is discharged from the bottom. This statement does not agree well with the data in Tables 121 and 122, but the conditions

[1] "By the 'water-retaining capacity' of a soil is to be understood the volume of water which remains in the soil after first drying the soil, filling it with water, and allowing the excess water to drain away." ("Sewage Treatment," Dunbar, page 135.)

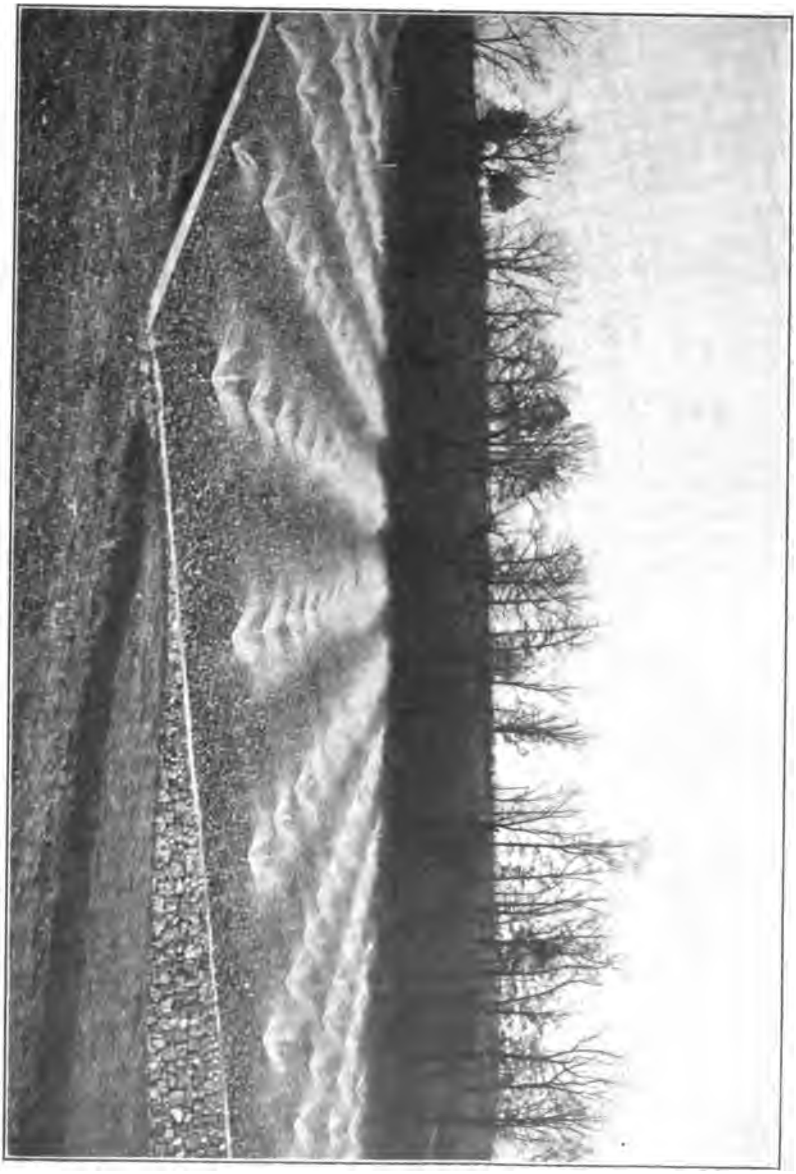


FIG. 131.—Trickling filter at Chambersburg, Pa.

TABLE 123.—TIME OF PERCOLATION OF SEWAGE THROUGH TRICKLING FILTERS

(William Clifford, *Proceedings Institution of Civil Engineers*, volume clxxii, part ii, page 3)

Grade	Rate of sprinkling		Average time of percolation		Interstitial water, gal. per cu. yd. (I)	$C = \frac{I}{RT}$
	Mil. U. S. gal. per acre per day	U. S. gal. per sq. yd. per hour (R)	2.4 ft. of medium, minutes	3 ft. of medium, minutes (T)		
Coal:						
¾ to ⅝ in....	1.39	11.952	24.3	30.4	11.40	0.0313
	1.05	9.000	31.7	39.6	10.56	0.0296
	0.61	5.256	45.6	57.0	9.84	0.0327
½ to ⅜ in....	1.78	15.852	26.4	33.0	14.64	0.0289
	0.78	6.696	49.4	61.8	14.16	0.0341
⅜ to ¼ in....	1.32	11.484	34.0	42.5	17.28	0.0357
	1.11	9.552	39.0	48.8	17.16	0.0366
	0.75	6.456	63.6	79.5	16.32	0.0317
¼ to ⅛ in....	1.37	11.796	54.7	68.4	19.26	0.0249
	1.05	9.048	64.3	80.4	18.84	0.0260
Gravel:						
1 to ¾ in.....	1.42	12.252	15.0	18.8	9.12	0.0396
	1.16	9.996	17.6	22.0	9.00	0.0409
	0.89	7.704	22.0	27.5	8.88	0.0417
¾ to ⅝ in....	1.50	12.948	20.1	25.1	11.04	0.0340
	1.13	9.696	31.5	39.4	10.80	0.0283
	0.80	6.852	35.8	44.8	10.08	0.0328
⅝ to ½ in....	1.45	12.504	31.2	39.0	11.70	0.0241
	1.12	9.600	37.2	46.5	11.40	0.0255
	0.96	8.304	40.0	50.0	11.16	0.0269
½ to ¼ in....	1.43	12.300	33.7	42.1	18.12	0.0350
	1.14	9.852	40.0	50.0	17.55	0.0356
	0.90	7.752	44.1	55.1	17.04	0.0399
Destructor clinker:						
1 to ¾ in.....	1.34	11.496	27.1	33.9	12.84	0.0332
	1.16	9.996	30.0	37.5	12.60	0.0337
	0.90	7.776	36.8	46.0	12.12	0.0338
¾ to ⅜ in....	1.45	12.504	47.1	58.8	19.44	0.0265
	1.16	9.996	60.9	76.3	18.72	0.0246
	0.85	7.344	84.7	105.9	18.24	0.0235
⅜ to ⅛ in....	1.43	12.300	120.0	150.0	28.92	0.0157
	1.16	9.996	148.0	185.0	28.20	0.0153
Average..........						0.0307

of the experiments appear to have differed. However, if the action of the trickling filter is dependent upon absorption, sufficient opportunity must be afforded the sewage matters to come in contact with and be retained by the absorbing medium.

William Clifford carried out experiments with trickling filters of coal, gravel and clinker from refuse destructors, with the results given in Table 123. The last column of this table gives a constant, c, which Clifford used in a formula to give the time of percolation, T, in minutes, of all particles of a dose through a filter. This formula is $T = cI/R$, where I is the water not held by the water-retaining capacity of a bed drained 20 hours, and R is the rate of sprinkling in U. S. gallons per square yard per hour. He summed up his investigation as follows:

"The time of percolation through clean filtering material varies inversely as the rate of sprinkling and directly as the amount of water taking part in the water movement through the bed, the amount of water in motion being generally represented by the interstitial water." (*Proc. Inst. C. E.*, vol. clxxii, Part II, page 9.)

TABLE 124.—PERCOLATING FILTERS OF EQUIVALENT CAPACITY BASED ON A TIME OF PERCOLATION OF 100 MINUTES

(Clifford, *Proc. Inst. C. E.*, volume clxxii, part ii, page 11)

Gravel			Coal			Slag			Clinker		
Grade in. in.	Depth ft.	in.	Grade in. in.	Depth ft.	in.	Grade in. in.	Depth ft.	in.	Grade in. in.	Depth ft.	in.
1 –3/4	8	3	3/4–5/8	7	2	1 1/4–3/4	6	1	1–3/4	6	1
3/4–5/8	7	3	1/2–3/8	5	2	3/4–5/8	5	4	5/8–3/8	3	9
5/8–1/2	6	7	3/8–1/4	4	5	5/8–3/8	4	9	....	.....	
1/2–1/4	4	4	1/4–1/8	4	0	3/8–1/4	4	0	....	.....	
....	....		....	....		1/4–1/8	3	10	....	.....	

By interstitial water Clifford referred to the quantity which will leave a filter in 20 hours of draining from a filled state. He calculated the depth of filters of several materials of different sizes to accomplish the purification of a sewage requiring a time of percolation of 100 minutes to be treated properly, using the formula already given for calculating the quantity of interstitial water, and the following formula for the depth required:

$$\text{Depth of bed in yards} = \frac{\text{total interstitial water}}{\text{interstitial water per cubic yard}}$$

The results of these computations are given in Table 124, and show

the effect of reducing the size of the medium in diminishing the necessary depth of the filter. It is probable that some of the variations in depth given in the table are too great, because the material was clean instead of being coated with bacterial jelly, as in practice. As small material should be used as is consistent with abundant air supply and ability to unload from time to time the solids which have been retained in the filter and transformed in character.

It should not be assumed that in a trickling filter oxygen performs its only work through solution or absorption by the sewage, for Dunbar demonstrated the fact that the bacterial jelly absorbs oxygen with great avidity.

"In order to demonstrate the absorbent powers of this film, I have washed out small quantities from biological filters, placed the material in closed bottles provided with manometers, and admitted known volumes of absorbable gases, such as oxygen and carbon dioxide. After a very short time the manometers showed a diminution in pressure, an increasing diminution being observed as the gases were absorbed by the sludge." ("Principles of Sewage Treatment," page 143).

"After the sewage has passed through the filter, *i.e.*, after a few seconds or minutes, according to the size and depth of the filtering material, the gases of the atmosphere enter the pores of the material. The retained organic matters are decomposed with the aid of micro-organisms. The absorbed oxygen is thus used up and replaced by fresh oxygen from the air in the pores. This causes a partial vacuum in the pores of the filter, and the surrounding air is drawn in with considerable energy. By means of an experiment I was able to show that the oxygen was drawn in, even through very narrow glass tubes from a closed vessel connected with the filter, with such energy as to cause a considerable diminution of pressure in the vessel. The oxygen then oxidizes the organic matters which have been retained by the filter and broken down by the action of micro-organisms. The process is accompanied by the production of considerable volumes of carbon dioxide, which is partly absorbed, partly leaves the filter with the next charge of sewage, and partly escapes by gaseous diffusion. The atmosphere over a biological filter contained 0.6 per cent. of carbon dioxide, *i.e.*, twenty times as much as is present in the atmosphere normally." (*Ibid.*, page 144.)

In order to unload transformed solids, it is important for the filtering material to be not only sufficiently coarse but also reasonably uniform in size. If the pores of coarse particles are filled with fine material, the solids will be retained in the bed indefinitely and clogging will eventually ensue. The more uniform in size the material is, the smaller it can be, safely. Much difficulty has been experienced in practice in securing material within the limits of size specified. Unless fine material can be excluded at the crusher, the material should be rescreened

before placing it in the filter. The size of material in a number of American trickling filters is given in Table 125.

TABLE 125.—SIZE AND KIND OF MATERIAL USED AT TRICKLING FILTER PLANTS IN THE UNITED STATES

Location	Kind of material	Size of material, inches
Columbus	Broken stone	1–3
Reading	Broken slag	1½–4
	Broken feldspar	2½
Gloversville	Broken limestone	1–2½
Baltimore	Broken stone	1–2½
Atlanta	Broken stone	1½–2½
Fitchburg	Broken granite	1–2

It is quite generally agreed that sewage should be fairly well settled preliminary to treatment by trickling filters in order to reduce the danger of clogging to a minimum. The more thorough the preliminary removal of suspended matters from the sewage the finer the filtering material which can be employed with safety.

DEPTH OF FILTERS

At Baltimore, experiments were conducted to ascertain the best size of material and depth of filter. Filters from 4½ to 12 ft. deep were operated at uniform rates for a period of time so short that caution should be exercised in interpreting the results. Hendrick stated:

"This figure brings out very strikingly the fact that the amount of purification per foot of bed decreases as the bed increases in depth, with the exception of nitrification, which seems to vary almost directly as the depth of the bed. . . . In the case of Baltimore, where it was required that the highest practicable degree of purification be obtained, the desirable depth . . . has been decided to be 9 ft. . . . There is considerable increase of purification between the depths of 6 and 9 ft., while the increased purification between depths of filters of 9 ft. and 12 ft., is relatively small." (Report of Baltimore Sewerage Commission, 1911, page 59.)

These conclusions were based partly on the average relative stability of the effluents.

Many of the effluent samples, particularly in the case of the deeper filters, had a stability above 100 per cent. Inasmuch as the putrescibility test does not indicate values exceeding 100 per cent., average relative stability numbers made up from results where the samples are partly above and partly below 100 per cent. stability are too low and do not represent the true character of the average effluent.

It is impossible to demonstrate the most economical depth of filter from the varying degrees of purification effected by filters of several depths dosed at a uniform rate per acre. On the other hand, if similar filters of different depths are dosed with the same quantities of sewage per cubic yard of filtering medium it is a simple matter to determine the relation of depth to filter efficiency. The relation of the rate of application per acre to the rate per cubic yard of filtering material is shown in Table 126.

TABLE 126.—EQUIVALENT RATES OF FILTRATION EXPRESSED IN GALLONS PER ACRE AND IN GALLONS PER CUBIC YARD

Gallons per acre	Gallons per cubic yard for depths given in column headings					
	5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.
500,000	62	52	44	39	34	31
1,000,000	124	103	89	78	69	62
1,500,000	186	155	133	116	103	93
2,000,000	248	207	177	155	138	124
2,500,000	310	259	221	194	172	155
3,000,000	372	310	266	233	207	186

In the case of similar sewages, the load upon filters may be expressed in grams of nitrogen per cubic yard of filtering material. Data are brought together in Table 127 to show the relation of the stability of trickling filter effluents to the depth of filter when approximately uniform quantities of unoxidized nitrogen in the form of organic nitrogen and free ammonia were applied per cubic yard of filter. The Gloversville results indicate that the 10-ft. filter, although treating a greater quantity of nitrogen per cubic yard, produced nearly as good an effluent as the 7-ft. filter and a better effluent than the 5-ft. filter. The Worcester and Lawrence results indicate that purification was approximately proportional to the depth.

The Royal Commission on Sewage Disposal reached the conclusion quoted on page 27, that the purification effected by a given volume of materia was not affected by the depth.

Four trickling filters having 4, 6, 8 and 10 ft. of ¾ to 1½-in. stone were put in operation at the Lawrence Experiment Station in May, 1913. An attempt was made to operate them at rates giving effluents with about 15 parts per 1,000,000 of nitrates, for earlier experience had shown that this nitrification was necessary to have effluents of Lawrence filters stable. The filters reached a steady operating condition early in 1914, and the results obtained with them are given in Table 128, from a

TABLE 127.—WORK ACCOMPLISHED BY TRICKLING FILTERS OF DIFFERENT DEPTHS

	Depth of filter, feet	Size of stone, inches	Applied nitrogen, grams per cubic yard	Stable samples, percentage	Character of applied sewage	Period of operation
Gloversville[1]..	5	1½–2	6.90	32.2	Septic	54 days
Gloversville. .	7	1½–2	5.96	100.0	Septic	50 days
Gloversville. .	10	1½–2	6.95	91.6	Septic	252 days
Worcester[2]. . .	5	¾–2½	10.52	70.0	Settled	1 year
Worcester[2]. . .	7½	¾–2½	10.02	93.0	Settled	1 year
Worcester[3]. . .	7½	½–1½	11.00	100.0	Settled	100 days
Worcester[4]. . .	10	¾–2½	10.40	88.9	Imhoff t.	4 mo.
Worcester[5]. . .	10	½–1½	10.40	100.0	Imhoff t.	4 mo.
Lawrence[6]. . . .	5	¼–1	25.7	40.78	Unsettled	3 years
Lawrence[6]. . . .	8	¼–1	16.6	90–100	Unsettled	3 years
Lawrence[6, 7, 8]	8	¼–1	25.4	90–100	Settled	4 years
Lawrence[6]. . . .	10	¼–1	16.4	100	Unsettled	1 year
Lawrence[6]. . . .	10	¼–1	28.2	100	Settled	2 years
Lawrence[7]. . . .	10	¼–1	29.7	70	Settled	1 year
Lawrence[6]. . . .	10	¼–1	22.6	100	Unsettled	3 years
Lawrence[6, 7, 8]	10	¼–1	19.1	100	Settled	3 years
Lawrence[6]. . . .	10	¼–1	17.0	100	Settled	1 year
Lawrence[6, 7, 8]	10	¼–1	18.7	100	Settled	4 years

[1] Report on sewage disposal, Eddy and Vrooman.
[2] Report of sewer department, 1910, page 748.
[3] Report of sewer department, 1909, page 1049.
[4] Report of sewer department, 1911, page 511.
[5] Report of sewer department, 1911, page 510.
[6] Report of Mass. St. Bd. Health, 1908, page 378.
[7] Report of Mass. St. Bd. Health, 1909, page 298.
[8] Report of Mass. St. Bd. Health, 1910, page 258.

TABLE 128.—AVERAGE CHEMICAL ANALYSES OF EFFLUENTS FROM DECEMBER, 1913, TO SEPTEMBER, 1914, INCLUSIVE

(Parts per 100,000)

Depth of filter	Quantity of settled sewage applied daily		Ammonia			Chlorine	Nitrogen as		Oxygen consumed	Percentage of stability
	Gal. per acre	Gal. per cubic yard	Free	Albuminoid: Total	Albuminoid: In solution		Nitrates	Nitrites		
4 Ft.	332,700	51.7	1.8600	.3050	.2006	13.20	1.63	.0065	1.89	85
6 Ft.	585,100	60.6	1.2900	.2950	.1796	13.30	1.81	.0128	1.79	91
8 Ft.	1,801,000	139.8	1.3575	.3715	.2292	12.98	1.66	.0126	2.17	88
10 Ft.	3,733,000	231.8	1.7275	.3900	.2616	13.48	1.76	.0081	2.38	86

paper by H. W. Clark before the Boston Society of Civil Engineers. (*Jour. B. S. C. E.*, vol. ii, 1915, page 37.) These results indicate that stable effluents can be produced at higher rates per unit volume of material as the depth of filter is increased. Clark found later by chlorine tests of effluents from salted sewage that the time of contact in filters of different depths of the same material increased more rapidly than the increase in the depth of the filter.

In most cases where sufficient head is available, the cost of trickling filters per unit of volume decreases with the increase in depth, owing principally to the reduced cost of floor, underdrainage and distribution with deeper beds. If the quantity of sewage which can be satisfactorily purified by trickling filters is proportional to the depth, the deeper filters will be the more economical. In a paper by one of the authors (*Jour. Bos. Soc. C. E.*, February, 1915), Table 129 is given, showing the costs of trickling filters of different depths per cubic yard of effective filtering material, based on contract prices at Fitchburg, Mass.

TABLE 129.—COST PER EFFECTIVE CUBIC YARD OF TRICKLING FILTERS OF VARIOUS DEPTHS, BASED ON FITCHBURG PRICES

Depth of stone in feet	6	7	8	10
Assumed value of stone per cubic yard	1.0	1.0	1.0	1.0
Relative volumes of stone	1.0	1.0	1.0	1.0
Relative areas of filtering surface	1.0	0.857	0.75	0.60
Relative extent of walls (area)	1.0	1.08	1.15	1.29
Floor at $1.30 per cubic yard 6-ft. filter (varies as area).	$1.30	$1.11	$0.98	$0.78
Walls at 14½ cts. per cubic yard 6-ft. filter (varies as walls).	0.15	0.16	0.17	0.19
Underdrains and gallery at 34 cts. per cubic yard 6-ft. filter (varies as area).	0.34	0.29	0.26	0.20
Distribution at 80 cts. per cubic yard 6-ft. filter (varies as area).	0.80	0.69	0.60	0.48
Stone at $1.95 per cubic yard (varies as volume of stone).	1.95	1.95	1.95	1.95
Total	$4.54	$4.20	$3.96	$3.60

By applying these unit prices to filters of varying depths assumed to be capable of accomplishing the same purification per cubic yard of filtering material, the data in Table 130 were obtained.

The unit prices for construction will vary greatly in different localities and the amount of excavation required may materially modify the rela-

tive costs of filters of different depths. Where the available head is limited it will undoubtedly be less expensive to provide a larger area of relatively shallow beds and thereby avoid the cost of pumping. The estimated costs of construction of trickling filters of different depths for several localities are shown in Table 131.

TABLE 130.—COST OF SQUARE TRICKLING FILTERS OF VARIOUS DEPTHS TO DO THE WORK OF A 1-ACRE FILTER 6 FEET DEEP

Depth of stone in feet	6	7	8	10
Assumed value of stone per cu. yd.	1.0	1.0	1.0	1.0
Relative volumes of stone	1.0	1.0	1.0	1.0
Relative areas of filtering surface	1.0	0.857	0.75	0.60
Relative extent of walls (areas)	1.0	1.08	1.15	1.29
Floor system at $2.60 per sq. yd.	$12,580	$10,780	$ 9,440	$ 7,540
Walls at $2 per sq. yd. (718 sq. yd. in 6-ft. filter).	1,436	1,550	1,650	1,852
Underdrains and gallery at 68½ cts. per sq. yd. (area).	3,315	2,840	2,480	1,988
Distribution system at $1.60 per sq. yd. (area).	7,773	6,666	5,830	4,666
Stone at $1.95 per cu. yd.	18,880	18,880	18,880	18,880
Total, exclusive of engineering and excavation.	$43,984	$40,716	$38,280	$34,926

In a discussion of the paper in which these tables were given, George W. Fuller stated (*Jour. Bos. Soc. C. E.*, February, 1915, page 76) that investigations at Baltimore, Md., and Hanley and Leeds, England, convinced him that a greater purification was effected per cubic yard of stone by shallow filters. There were rather marked differences in the unit costs of floors, false bottoms, walls, galleries and distributors as given in Tables 129 and 130 and those employed by him, owing to local influences on design and market prices. His conclusion was that a trickling filter not less than 6 ft. nor more than 7 ft. deep will prove most economical in most cases. A leading reason for this difference in opinion lies in the methods employed in measuring the efficiency of such a filter. The strength of the sewage, S, in the investigations upon which the tables are based was measured in terms of unoxidized nitrogen, including free ammonia and organic nitrogen, whereas Fuller used the McGowan formulas

S = 4.5 (Ammonia N + Organic N) + 6.5 (Oxygen absorbed in 4 hours)

for sewage and the following for effluent:

$$S = 4.5\ (\text{Ammonia } N + \text{Organic } N) + 2\ (\text{Volatile suspended solids}) - 3\ (\text{Nitric nitrogen}).$$

In these formulas the nitrogen and oxygen are expressed in parts per 100,000.

TABLE 131.—APPROXIMATE COMPARATIVE COSTS OF FILTERS OF DIFFERENT DEPTHS BASED ON CONTRACT PRICES AND ESTIMATED QUANTITIES

	5-foot depth (2 acres)		7½-foot depth (1.333 acres)		10-foot depth (1 acre)	
	Cost of constr.	Per cent. of total	Cost of constr.	Per cent. of total	Cost of constr.	Per cent. of total
Gloversville, N. Y. prices:[1]						
Earthwork	$5,082	8.0	$5,420	9.8	$5,660	11.0
Walls	1,976	3.1	2,420	4.4	2,770	5.5
Distribution system	9,080	14.2	6,050	11.0	4,540	9.0
Collection system	19,708	30.9	13,180	24.0	9,854	19.4
Stone	27,932	43.8	27,932	50.8	27,932	55.1
Total	$63,778		$55,002		$50,696	
Columbus, Ohio, prices:						
Earthwork	$2,615	5.7	$3,630	8.8	$4,140	10.7
Walls	2,240	4.8	2,090	5.1	2,020	5.2
Distribution system	5,540	12.0	3,690	8.9	2,760	7.2
Collection system	12,254	26.5	8,160	19.8	6,125	15.8
Stone	23,600	51.0	23,600	57.4	23,600	61.1
Total	$46,249		$41,170		$38,645	
Washington, Pa. prices:						
Earthwork	$1,955	3.0	$1,878	3.8	$2,060	4.5
Walls	3,190	5.0	3,920	7.6	4,520	9.8
Distribution system	13,920	21.8	9,290	17.9	6,960	15.1
Collection system	25,000	39.1	16,650	32.2	12,500	27.2
Stone	19,900	31.1	19,900	38.5	19,900	43.4
Total	$63,965		$51,638		$45,940	
Fitchburg, Mass. prices:						
Earthwork	$6,720	8.1	$6,220	9.2	$5,982	10.0
Walls	1,583	1.9	1,738	2.6	1,821	3.0
Distribution system	15,550	18.7	10,380	15.3	7,773	12.9
Collection system	29,412	35.4	19,600	28.9	14,706	24.5
Stone	29,785	35.9	29,785	44.0	29,785	49.6
Total	$83,050		$67,723		$60,067	

[1] Roof omitted.

The depth of a trickling filter is limited by the requirements of proper ventilation, for if the oxygen of the air passing through the filter is

used up before the bottom is reached the lower part of the filter will be ineffective. For treating unusually strong sewages or industrial wastes having marked avidity for oxygen, trickling filters should not be as deep as for treating ordinary domestic sewage. Furthermore, there will be a greater tendency toward surface clogging and consequent obstruction of the circulation of air in deep beds on account of the greater quantity of sewage applied per unit of superficial area. This consideration is governed largely by the character and size of the filtering material and the efficiency of preliminary clarification. Sewage should be well settled before treatment by trickling filters. Organic growths have appeared at times on some trickling filters, more or less completely clogging the surface layer. Uneven distribution will more quickly result in clogging overdosed areas in the case of deep beds because of the higher rate of application per unit of area. If portions of the filter area must be rested from time to time on account of surface clogging, a larger proportion of the filter will necessarily be out of use in the case of deep beds.

DISTRIBUTION OF SEWAGE

Motion of Sewage in Bed.—Experiments by W. G. Taylor with filtering material of different sizes and depths showed that the lateral diffusion of percolating liquid was greater with fine than coarse material. With 2-in. crushed gneiss in a 6-ft. bed, 90 per cent. of the liquid applied at the surface in drops was not diffused beyond the circumference of a circle of 7-in. radius with its center at the point of application. With ⅛-in. stone, the diffusion in a 6-ft. bed did not exceed 12 in. from the center. The filtering material was clean, nearly uniform in size and in a moist but non-dripping condition at the beginning of each test. In most cases about one-half of the total lateral diffusion measured at the bottom of the bed took place in the top foot. Experiments with several points of application of the sewage to the same bed showed that where the cones of diffusion became joined the increase in the quantity of liquid produced a streaming effect, which was much more marked with high than low rates of application. Taylor's conclusion from the investigation was:

> "The coarse filling material ordinarily used in percolating filters does not tend to markedly equalize irregularities in the surface distribution; frequently the distribution at the base of a filter is much more irregular and imperfect than at the surface . . . and a high efficiency in subsurface distribution is fostered by a slow, continuous rate of application rather than by an intermittent application at a higher rate." (*Engineering Record*, vol. lix, June 5, 1909, page 711.)

Practically the same conclusions were reached by Clark and Gage from investigations at the Lawrence Experiment Station (1908 Report

Mass. St. Bd. Health, page 393) and by John D. Watson from investigations at Birmingham ("Works of Birmingham, Tame & Rea District Drainage Board," page 59.)

In practice accumulations of solids in a filter cause some lateral movements of the sewage and may actually improve the distribution by its retarding effect upon the passage of the sewage through the bed. On the other hand, if the clogging causes pooling at different points, the sewage will be diverted to the cleaner areas and streaming will occur, seriously interfering with uniformity of distribution.

Since little may be expected in the way of lateral distribution below

FIG. 132.—Stoddart dripping tray.

the surface, the importance of uniformity of surface distribution is apparent. The ideal to be attained may be illustrated by a rainstorm of sufficient intensity to produce the desired rate of application. A rate of 1,000,000 gal. per acre daily would be equivalent to a rainfall of about 1.5 in. per hour.

Dripping Tray.—The dripping tray, Fig. 132, devised by F. Wallis Stoddart of Bristol, England, consists of a series of corrugated sheet-iron troughs from which the sewage overflows continuously through perforations in the ridges between the channels, dripping from points on the lower side. Frequent brushing is required to keep the perforations clear. This distributor obstructs the free access of light and air

to the bed, which is a disadvantage, but it reduces the diffusion of odors. Another advantage is that very little head is required.

Revolving Distributors.—Traveling distributors are used extensively in Great Britain and at many of the Canadian trickling filters. There are 2 general types, one consisting of 2 or 4 horizontal arms revolving about a central post and used to dose circular beds, and the other consisting of a low truss spanning a rectangular bed and moving back and forth on its side walls while distributing liquid over the bed in various ways, according to the type of machine.

The revolving distributor used for dosing 2 filters 30 ft. in diameter shown in Fig. 133, is typical of small English plants. There is a revolving distributor 130 ft. in diameter at Malvern and others from 70 to 120 ft. in diameter have been constructed for different places, but most installations have probably had diameters under 70 ft. In the apparatus shown in Fig. 133 the weight is carried by a ball-bearing at

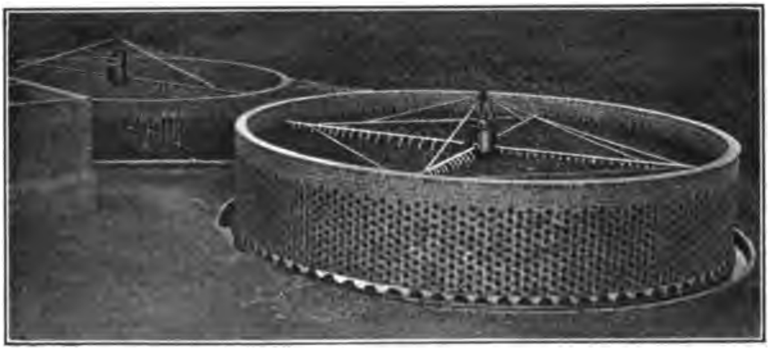


FIG. 133.—Revolving distributors of small English trickling filters.

the top of the vertical standard, Fig. 135. The reason for the small loss of head in such apparatus can be explained by outlining its operation. The base A is connected to a riser pipe in the center of the bed, by which the liquid is admitted to the distributor. In starting the apparatus, air is exhausted from the annular siphon H through the valve G by means of an air pump, thus starting the flow of liquid from A into the revolving cylinder D, from which the perforated arms E extend over the bed. The siphon is disconnected from the supply pipe by the fixed cylinder C and the ports F. Without this fixed disconnecting cylinder the supply pipe would form part of the siphon and several inches of head would be lost. The liquid flows out of the holes on the sides of the arms, and the reaction of all these little jets causes the apparatus to revolve about the ball-bearing on top of the post. Swaying is prevented by a gun-metal bearing N. Both these bearings run in oil. The head lost in such an apparatus is very small.

Occasionally the weight of the revolving arms is carried by small trucks running on tracks on the surface of the bed, Fig. 167.

In Fig. 136 is shown a typical standard for a float-supported distributor. A revolving distributor for a 100-ft. bed will weigh over 2 tons, and it is desirable to have this weight supported so as to require the minimum amount of power for rotation. This purpose is accomplished by placing all the moving parts on a large float. If the distributor receives storm water as well as dry-weather sewage, two of the arms have small weirs at their inlets which concentrate the dry-weather flow in 2 arms and improve the operation of the machine in this way. The vertical movement of the distributor is limited by a thrust

FIG. 134.—Traveling distributor of water-wheel type (Fiddian).

bearing in the top of the standard, and the side thrust is taken up by a ball bearing in the mercury seal casting.

Traveling Distributors.—An entirely different principle is employed in the water-wheel type of distributors, Fig. 134, which seem to be particularly favored in England for dosing filters of fine material where particularly uniform distribution is desired. These distributors can be constructed in a revolving form or to travel back and forth as illustrated in Fig. 134. Either type is driven by the fall of the liquid through a height of about 18 in. With the rotating type the liquid is supplied through a hollow standard in the center of the bed, while in the traveling type it is siphoned out of a channel running along one side of the bed or between each pair of beds. The liquid passes through a feed

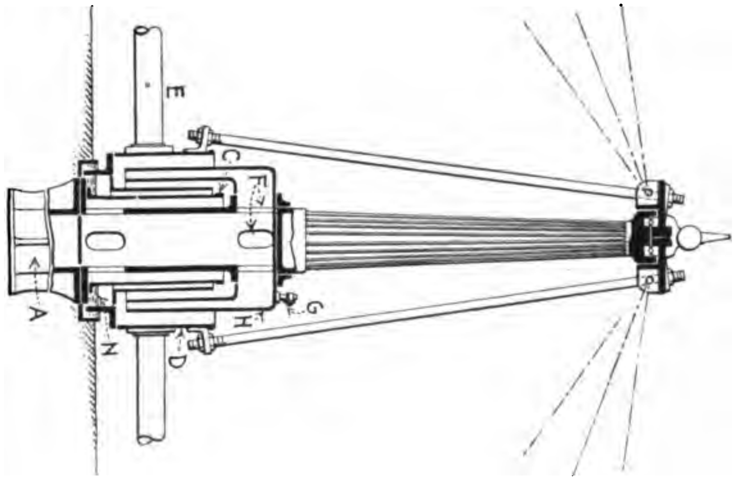


FIG. 135.—Standard for center-supported distributor (Jennings).

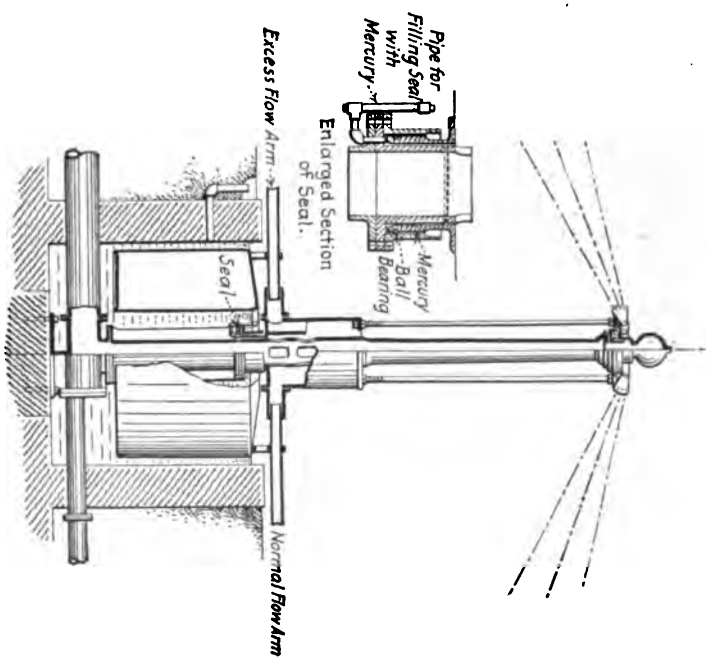


FIG. 136.—Standard for float-supported distributor (Candy-Whittaker).

tube running the whole length of the distributor. This tube has apertures through which the liquid drops into the buckets of a long water-wheel. The feeding arrangements on the distributor are reversed at each end of the bed by a buffer attached to the masonry, and as soon as the reversal occurs the apparatus begins its return trip. The beds shown in the illustration are 92 ft. long, 14 ft. wide and 3 ft. deep.

Traveling distributors of another type are used at the trickling filters at Springfield, Mo., constructed from the plans of Alexander Potter. They were adopted on account of the low operating head they required, only 12 in. There are six 200 × 53¾-ft. beds, over each of which a distributor is hauled back and forth by an endless wire rope. A 6-h.p. gasoline engine drives all the cables in one direction continuously, and by means of automatic reversing gear on each distributor it is attached alternately to the two runs which form each continuous cable. Settling tank effluent is siphoned from a distributing channel, in the same position illustrated in Fig. 136, and flows into 2 feed tubes 7 in. in diameter. Each has 4 × ⅝-in. orifices 15 in. apart through which the tank effluent passes to a 2½-in. tube, between the 2 feed tubes. This small tube can be raised or lowered in such a way as to control the distribution of the liquid over the bed. Sheet-iron covers are placed over the tubes and their sides extend to within 1 in. of the bed in order to conserve the heat of the sewage during cold weather. (*Proc. Am. Soc. Mun. Imp.*, 1913, page 34.) Potter informed the authors in May, 1915, that one operating advantage of the covered distributor was an almost total absence of flies around the beds. The only weak point found about the installation was the maintenance of the cables, concerning which he wrote:

"During the cold weather the cables sometimes become coated with ice and prevent the clutch on the traveling distributor from securing a proper grip. Last year, the man in charge of the plant replaced two of these cables with a ½-h.p. electric motor mounted directly upon the traveling distributor. The operation of these motors has given satisfaction during the last cold season, and at once does away with the cable mechanism with its clutches, pulleys, etc. It is the intention to equip all of the traveling distributors with similar small electric motors."

Spraying Nozzles and Dash Plates.—Another method of distribution, in use in England and employed almost exclusively in the United States, is to spray the liquid from a number of fixed nozzles. These were probably first used at Salford, England, where at first the spray was obtained by the impact of 2 converging streams, Fig. 137. The later form, Fig. 137, has openings designed to give a rotating movement of the stream. The Birmingham, England, nozzle, Fig. 137, has a ⅜-in. opening through which a 7⁄32-in. spindle passes, carrying

a plug which sprays the sewage. Nozzles with such small apertures require careful preliminary treatment of the sewage in order to avoid undue clogging of the nozzles. At Birmingham, in spite of thorough preliminary settling, 1 man was required for each 1½ acres to keep the nozzles free. On the most recent installation strainers for the filter influent have been provided, reducing the cost of maintenance nearly 65 per cent.

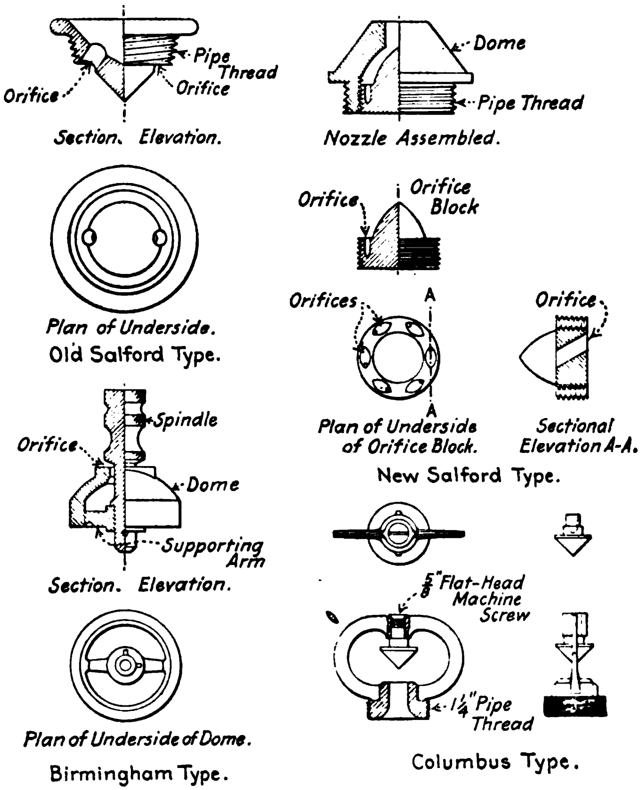


FIG. 137.—Types of nozzles for trickling filters.

The nozzle used at Columbus, Ohio, Fig. 137, has a clear orifice of 9/16 in., the distribution being effected by a cone supported by arms. This nozzle is less likely to clog than the Birmingham and Salford nozzles, but the arms interfere with the distribution, especially when they become coated with fungus growth. Modifications of this nozzle have been made by placing the distributing cone on a long swinging arm, which interferes little with distribution.

The Weand nozzle used at Reading, Pa., Fig. 138, somewhat resembles an inverted Columbus nozzle, the exterior arms being eliminated. The spraying cone is carried on a spindle 5/16 in. in diameter secured in place by a thread which fits into the lower part of the casting. The orifice is in the form of an annular ring, 5/16 in. wide, around the spindle. This type was modified for Atlanta, Ga., as shown in Fig. 138.

The gravity distributor used at the sewage experiment station of

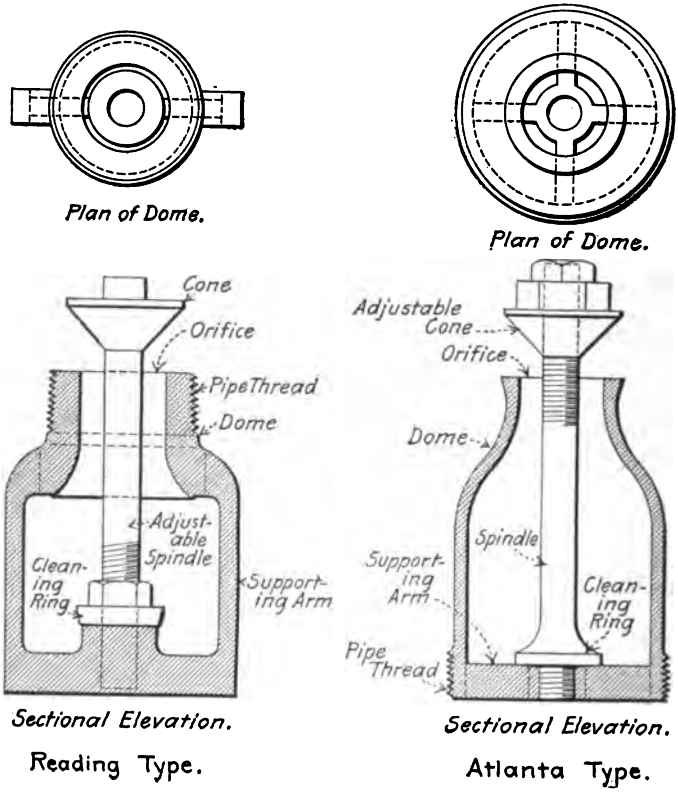


FIG. 138.—Two types of Weand nozzle.

the Massachusetts Institute of Technology discharged the sewage from openings in the bottom of troughs or from pipes onto concave metal disks placed beneath, which caused the sewage to splash upward and outward in the form of a spray. This type is in use on 1.2 acres of filters at Mt. Vernon, N. Y. Fig. 139 is another example of this type of distribution, designed by Thomas L. Fountain for use at Calvert, Texas. This method of distribution is better adapted for warm climates than for cold on account of the exposed distribution system.

The Taylor square spray nozzle (Fig. 140), designed by Taylor for Waterbury, Conn., has an orifice 1 in. in diameter, through which a spindle passes, carrying a four-lobed, spreading cone, designed to

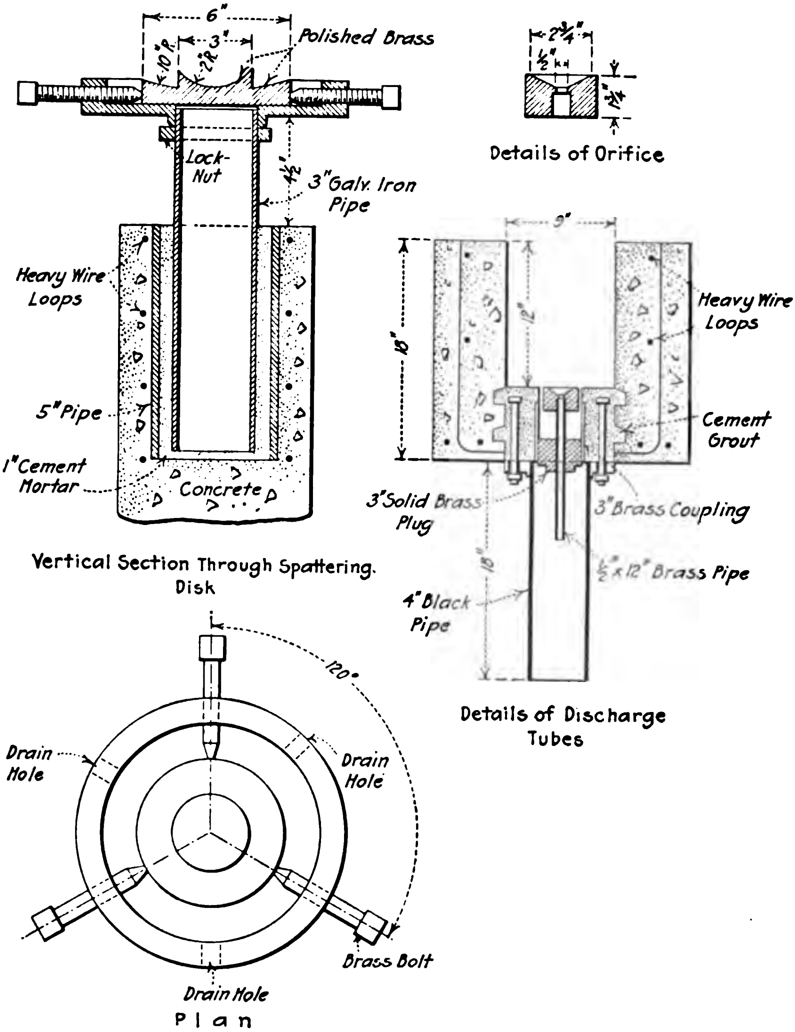


FIG. 139.—Splash plate and discharge tube, Calvert, Tex.

spray a square area. The distributing cones of such nozzles must, of course, be kept in a definite position in order to spray contiguous square areas. These nozzles are designed to concentrate the spray in a narrow

zone and their success depends upon a rapidly varying head. In order to accomplish this result Taylor has designed a pressure undulating valve described in Chapter XVIII. Taylor nozzles may also be obtained in the form of six-lobed cones designed to spray a hexagonal area, and in an unlobed form for circular sprays.

Another nozzle which is designed to spray a square area is the Merritt nozzle, Fig. 141. The top of this nozzle is in the form of a square rather than the four-leafed clover design of the Taylor nozzle. The interior supports for the spindle afford considerable opportunity for

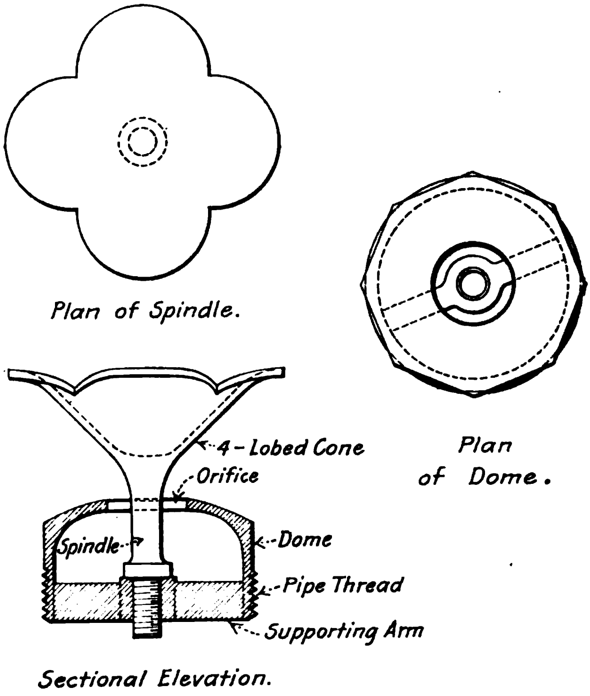


FIG. 140.—Taylor square-spray nozzle.

an accumulation of clogging matters although the nozzle may be readily cleaned after unscrewing it from the riser pipe. This nozzle is also designed in forms to spray either a hexagonal area or a circular area. Merritt square-spray nozzles are used on a large area at Baltimore, Md.

Still another form of nozzle designed to effect a square spray, Fig. 142, was devised by E. Sherman Chase at Reading, Pa. This nozzle has a square orifice with a spindle passing up through the center, carrying an inverted pyramid having the sides of its base at an angle of 45 deg. with the sides of the orifice.

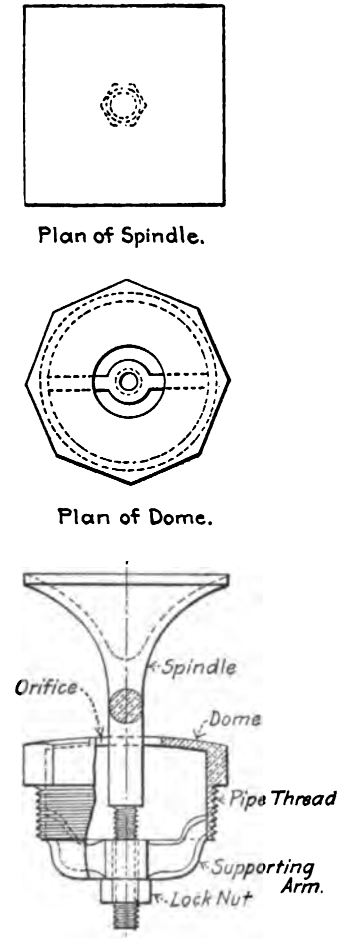


FIG. 141.—Merritt square-spray nozzle.

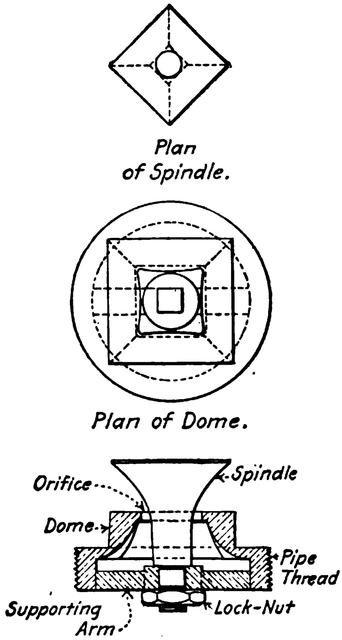


FIG. 142.—Chase square-spray nozzle.

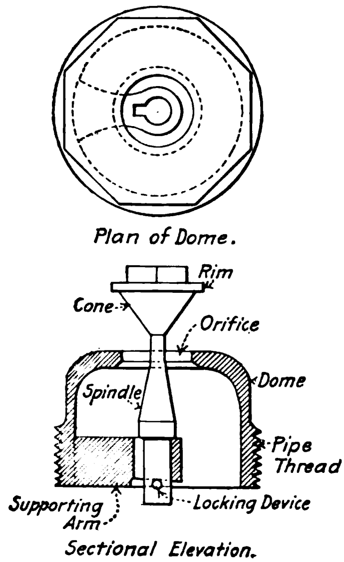


FIG. 143.—Worcester type of nozzle.

The nozzle developed at Worcester, Mass., Fig. 143, has at the base of the spraying cone a rim or lip, the function of which is to break up the sewage into a finer spray and spread it over a wider area. The spindle has a locking device so that it can be quickly removed when cleaning the orifice.

EFFICIENCY OF FIXED NOZZLES

The design of a nozzle should be such as to minimize clogging and to permit rapid cleaning when clogging does occur. Nozzles having small orifices generally produce a relatively fine spray, somewhat greater saturation of the sewage with atmospheric oxygen, and a more uniform distribution, but the cost of keeping them clear may be a large item. If the orifice is so large that the sewage is delivered to the bed in a sheet, the streaming effect may prevent efficient purification. More odor is probable with a fine spray and a greater reduction in the temperature of the sewage during cold weather.

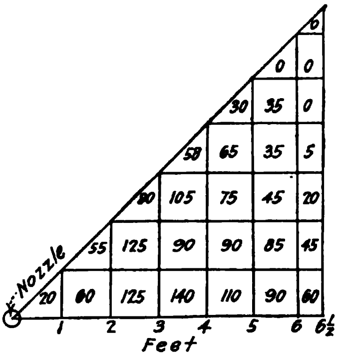


FIG. 144.—Results of test of Weand nozzle.

Diagram shows distribution in gallons per 24 hours of a Weand nozzle tested at Reading, Pa., by E. Sherman Chase. The dosing head dropped from 5.3 to 1.5 ft. in 3 min. 20 sec.; doses were intermittent and numbered 160 per day. Allowance was made for overlap.

Other conditions being equal, the nozzle which sprays the greatest area under a given head is the most economical. If this result is secured by a concentration of spray in a narrow ring, satisfactory distribution will depend upon a rapid variation in head during the discharge. The efficiency of practically all fixed nozzles may be greatly improved by a variation in head, because a comparatively large proportion of the discharge is concentrated upon a relatively narrow ring of the filter area. Methods of causing such variations are explained in Chapter XVIII.

At Reading, where the head is varied from 6 to 1.4 ft. by means of a tapered dosing tank, a distribution was secured by E. Sherman Chase which is shown in Fig. 144. The butterfly valves which were formerly used at Reading have been replaced by siphon dosing tanks, on account of unsatisfactory operation of the valves. One difficulty in the use of undulating valves lies in caring for the variation in sewage flow. This would in some cases make necessary the construction of equalizing tanks.

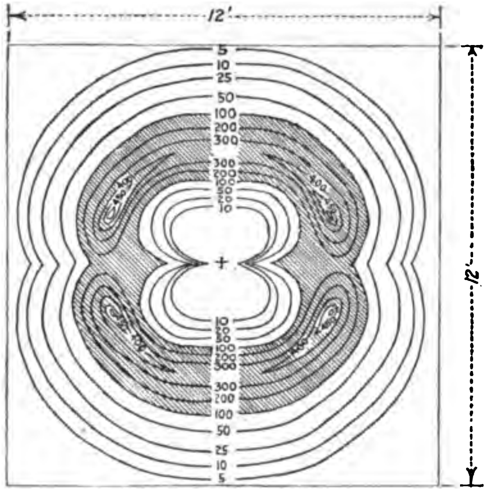


FIG. 145.—Distribution produced by a Columbus nozzle.
(Overdosed area, cross hatched; remainder, underdosed.)

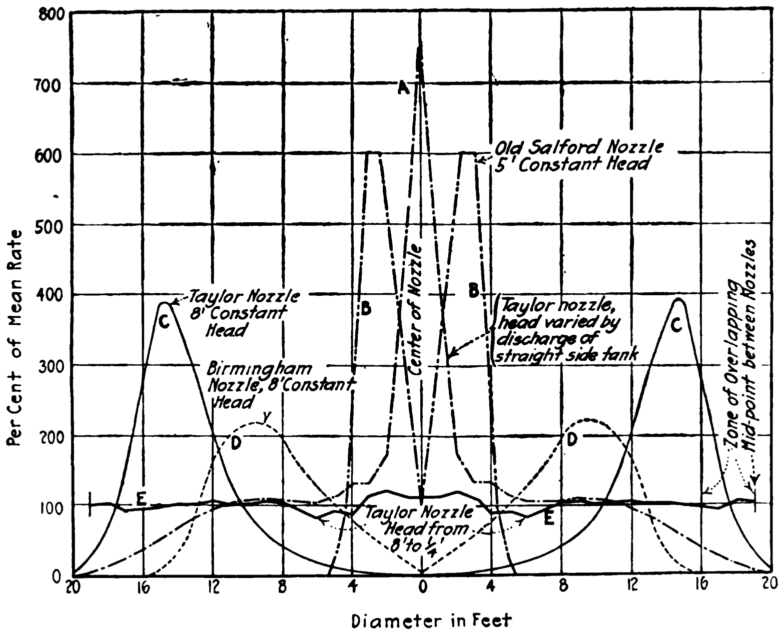
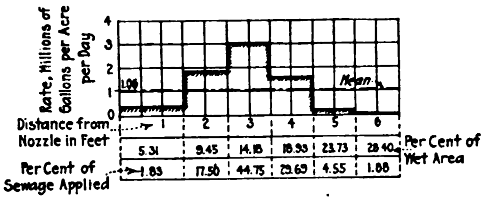


FIG. 146.—Distribution effected by several nozzles under specified conditions.

In tests of nozzles at the experiment station of the Massachusetts Institute of Technology[1] the spray was collected in a galvanized iron



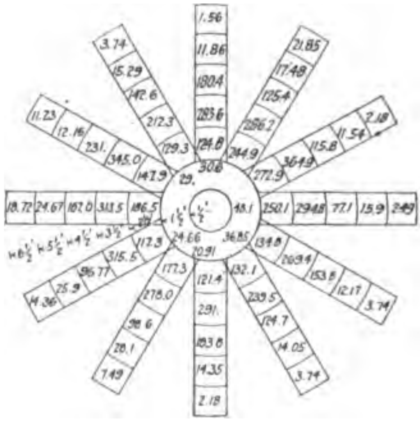


FIG. 147.—Record of a test of a Birmingham nozzle at Brooklyn, N. Y.
(Figures on radial arms are percentages of an evenly distributed spray.)

TABLE 132.—COMPARATIVE EFFICIENCY OF DISTRIBUTORS OF VARIOUS TYPES UNDER BEST CONDITIONS

(Winslow, Phelps, Story and McRae, *Technology Quarterly*, vol. xx, 1907, page 235)

Type	Rate, gal. per minute	Sprinklers per acre to discharge 2,000,000 gal. per day	Coefficient of distribution
Best gravity distributor.	4.08	341	0.76
Old Salford	2.9	483	0.44
New Salford	2.1	667	0.78
Birmingham	2.0	700	0.80
Columbus	14.8	94	0.61
Waterbury	10.4	134	0.73

[1] "Studies of Sewage Distributors for Trickling Filters," by C. E. A. Winslow, E. B. Phelps, C. F. Story and H. C. McRae. *Technology Quarterly*, Sept., 1907, xx, 325. Also "Contributions from the Sanitary Research Laboratory and Sewage Experiment Station of the Massachusetts Institute of Technology," vol. vi.

pan in the form of a 30-deg. circular sector of 6 ft. radius, divided into 6-in. compartments by concentric arcs. The pan could be rotated about the nozzle during the tests. The best results are summarized in Table 132. All nozzles were under 6 ft. head. The best gravity distributor had a dash plate 3 in. in diameter with the top surface curved to a 2-in. radius, and was placed 2 ft. above the surface of the bed. For tests of dash plates at Mt. Vernon, N. Y., see *Engineering News*, March 24, 1910.

The effect on the distribution of the Columbus nozzle caused by its

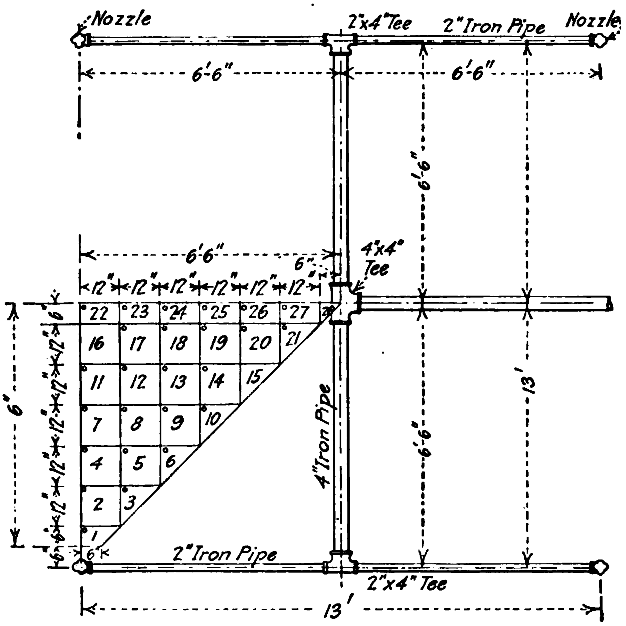


FIG. 148.—Pan for testing distribution by square-spray nozzles, Fitchburg, Mass.

side arms is shown in Fig. 145 (Gage, *Engineering News*, August 20, 1908). In such diagrams the distribution is shown by contour lines of equal quantities of water upon unit areas, expressed as percentages of the mean rate on the wet area. The results of several tests by Taylor are given in Fig. 146; all were made with the nozzle 6 in. above the collecting surface (*Engineering News*, Nov. 11, 1909). Tests made by E. J. Fort were recorded as shown in Fig. 147 in the 1908 report of the Brooklyn Bureau of Sewers. These results were obtained by a modified form of the Birmingham nozzle working under a 4-ft. head and discharging 32,400 gal. daily. The variation in circumferential distribution

as well as in radial distribution is given in percentages of an evenly distributed quantity.

A long series of careful tests was made at Fitchburg, Mass., under the direction of the authors, by F. W. Jones, chemist under David A. Hartwell, chief engineer of the Sewage Disposal Commission, to determine the uniformity of distribution with various types of nozzle working under a proper variation in head. A battery of nozzles was used in order to determine the actual effect of overlapping sprays. Four square-spray nozzles were arranged at the corners of a square area

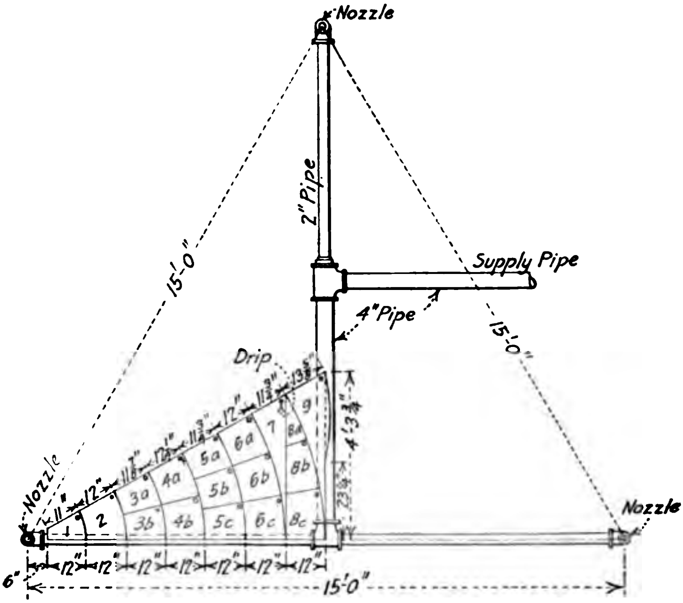


FIG. 149.—Pan for testing the distribution by circular-spray nozzles, Fitchburg.

and 3 circular-spray nozzles were placed at the apices of an equilateral triangle. Spacings of nozzles of 13 and 15 ft. and different heads between 2 and 10 ft. were studied.

The spray was collected in the compartments of pans, shown in place in Figs. 148 and 149. The pan had one-eighth of the area to which each nozzle was contributary in the tests of square-spray nozzles and one-twelfth of the area to which each circular spray nozzle was contributary. Each test was carried on for an exact period of time at a definite working head, and the amounts collected in each compartment were measured. The discharge of each nozzle under the different

heads was metered, and the effective head was determined in each case.

The relative period of operation required at each head to produce the most even distribution was computed, and the distribution that could be effected by the nozzle with different ranges in head was determined

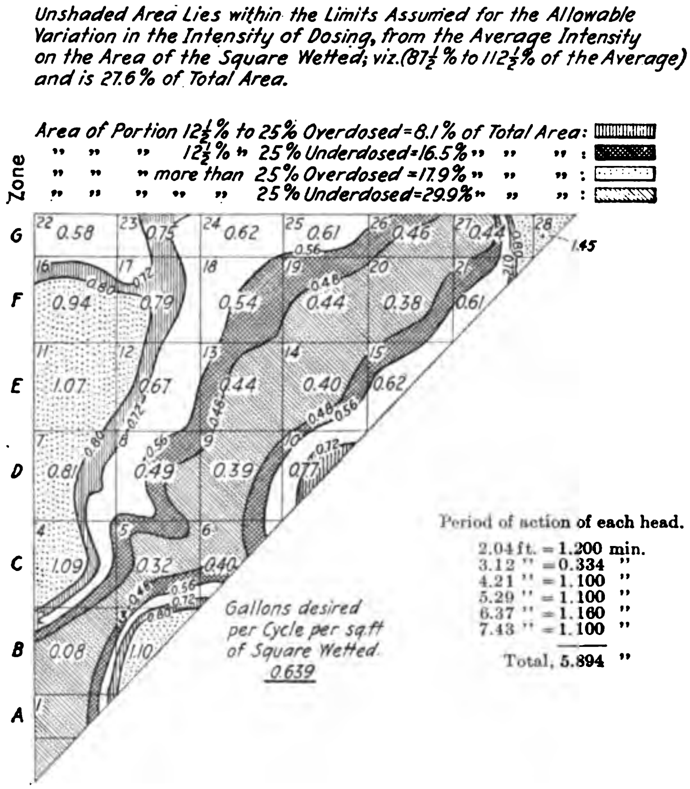


FIG. 150.—Estimated distribution in gallons per square foot per theoretical dosing cycle by Taylor square-spray nozzle, with 7/8-in. orifice and 13-ft. spacing.

The average operating heads range from 2.04 to 7.43 ft. and the period of action of each head is varied to produce an approximately uniform distribution in one cycle.

Letter of zone	A	B	C	D	E	F	G	Total
Area, sq. ft.	3.0	12.0	20.0	28.0	36.0	44.0	25.0	168.0

on that basis. It was assumed that on all areas receiving between 87½ per cent. and 112½ per cent. of the mean rate, the distribution was good. For the underdosed areas receiving 75 to 87½ per cent. of the mean rate and for the overdosed areas receiving 112½ to 125 per cent. of the mean rate, the distribution was called fair. The distribution on the remaining area was called poor, whether overdosed or underdosed.

The uniformity of distribution was studied by means of contour lines enclosing the areas of "good," "fair" and "poor" distribution.

A typical example of the distribution effected by the Taylor square-spray nozzle is shown in Fig. 150. The marked variation in distribution along different radial lines from the nozzle is striking. That the

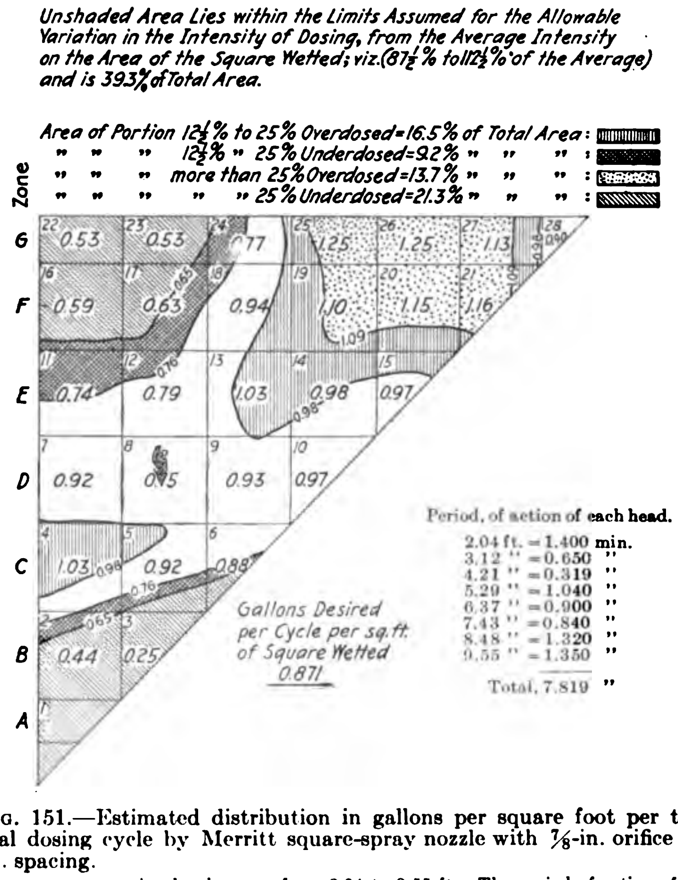


FIG. 151.—Estimated distribution in gallons per square foot per theoretical dosing cycle by Merritt square-spray nozzle with ⅞-in. orifice and 13-ft. spacing.

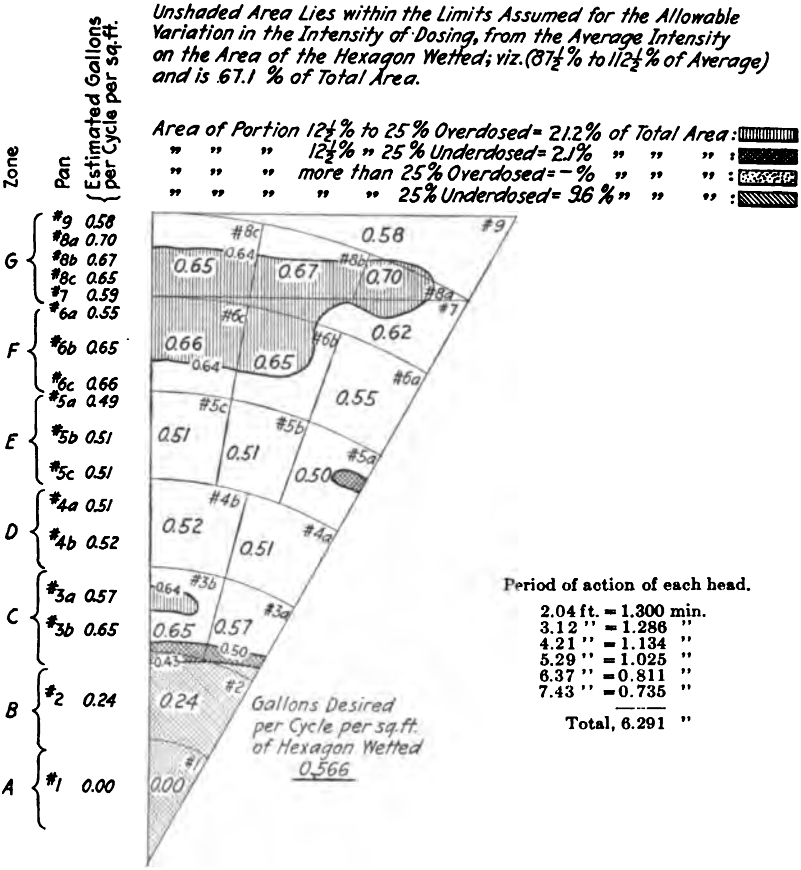
The average operating heads range from 2.04 to 9.55 ft. The period of action of each head is so varied as to produce an approximately uniform distribution in one cycle.

Letter of zone	A	B	C	D	E	F	G	Total
Area, sq. ft.	3.0	12.0	20.0	28.0	36.0	44.0	25.0	168.0

character of the distribution effected by the Merritt square-spray nozzle is entirely different from that of the Taylor square-spray nozzle is shown in Fig. 151. In this case there are large overdosed and underdosed areas remote from the nozzle. The large overdosed area was not due to excessive overlapping of nozzle sprays or too close spacing of the

nozzles, for with a lower maximum head, equivalent to a closer nozzle-spacing, a greater overdosed area resulted.

The improvement in uniformity of distribution with the circular-



Period of action of each head.

2.04 ft. =	1.300 min.
3.12 " =	1.286 "
4.21 " =	1.134 "
5.29 " =	1.025 "
6.37 " =	0.811 "
7.43 " =	0.735 "
Total,	6.291 "

Fig. 152.—Estimated distribution in gallons per square foot per theoretical dosing cycle by Taylor circular nozzle with ⅞-in. orifice and 15-ft. spacing.

The average operating heads range from 2.04 to 7.43 ft. The period of action of each head is varied to produce an approximately uniform distribution in one cycle.

Letter of zone	A	B	C	D	E	F	G	Total
Area, sq. ft.	6.16	12.60	18.30	25.55	30.60	38.22	62.84	194.27

spray Taylor nozzle over the square-spray type is illustrated in Fig. 152. An almost ideal distribution was given by the Worcester circular-spray nozzle. An experimental dosing-tank was constructed and further tests were made with it, with results shown in Fig. 153. While

this distribution was not so good as that indicated by the theoretical dosing cycle, there was no area badly overdosed and the only area badly underdosed was adjacent to the nozzle. The latter defect would be remedied largely by a lower minimum head. In practice, however,

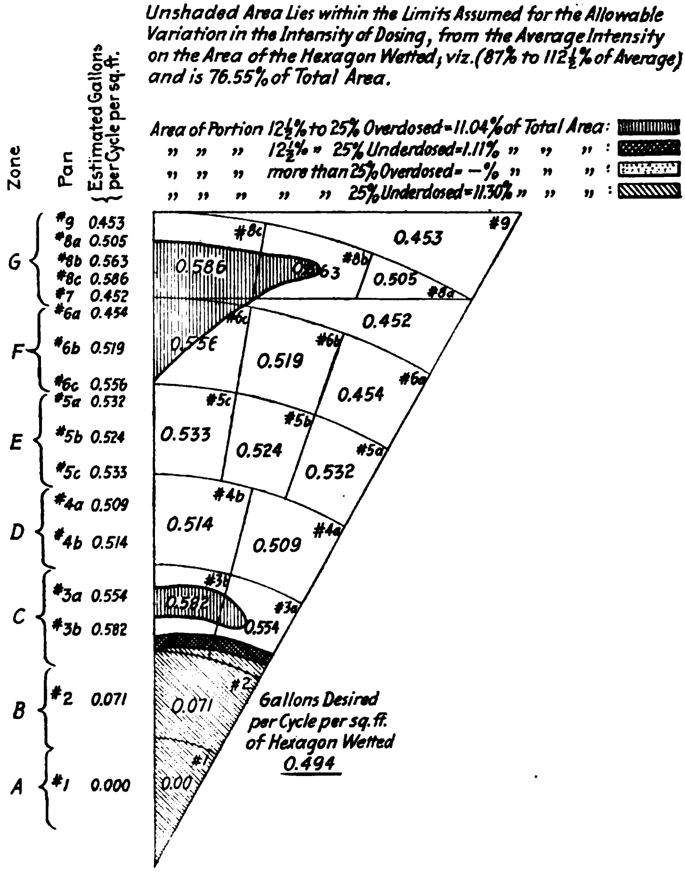


FIG. 153.—Distribution accomplished by Fitchburg experimental dosing tank using Worcester nozzle with $1\frac{3}{16}$-in. orifice and 15-ft. spacing.

The operating heads range from 2.00 to 7.96 ft. The quantities are in gallons per cycle per square foot. The time required to empty the tank was 5 min. 29 sec.

Percentage of desired dose	75	87½	100	112½	125
Gallons per square foot	0.371	0.432	0.494	0.557	0.618

there is likely to be a surplus of sewage in the distribution system after the siphon goes out of action, which will find its way on to this area. It is probable, also, that on account of the streaming effect produced by a fixed nozzle under very low head, it would be unwise

to apply as great a quantity of sewage to the area close to the nozzle as to the area more remote.

SPACING OF NOZZLES

To secure uniform distribution with fixed nozzles, it is essential to give careful consideration to spacing them. Circular-spray nozzles can cover only 71.5 per cent. of a rectangular area when placed at the centers of contiguous square areas dosed without overlapping of sprays from adjacent nozzles, *a*, Fig. 154. This fact led to the design of nozzles to spray a square area.

When circular-spray nozzles are arranged at the apices of equilateral triangles, *b*, Fig. 154, the loss in area undosed is reduced to 9.9 per cent. Nozzles have been designed to throw a hexagonal spray with a view to utilizing this waste area.

When circular spray nozzles are brought sufficiently close together so that the sprays from any three adjacent nozzles just touch at the point equidistant from them, *c*, Fig. 154, there is no unused area except around the outer edge of the filter. The nozzles are at the centers of circles circumscribed about interlocking hexagons. If the radius of spray, *ao*, be represented by R the nozzles will be spaced at intervals of $1.732R$, *be*, and the distance between the rows of nozzles, *ad*, will be $1.5R$.

By the use of nozzles throwing a semicircular spray, the unused portion or loss in area on the sides AD and BC, *c*, Fig. 154, can be materially reduced. The percentage loss in area is decreased as the area of filter is increased, and in the case of a large area becomes insignificant. While it will undoubtedly be advantageous to use nozzles throwing sprays in the form of half-circles, it is probable they will not prove as satisfactory in operation as those of full size. It would be unwise, therefore, to use them on the sides AB and CD for the sake of the theoretical reduction in loss of area.

It appears from Fig. 154 that a considerable area is overdosed, due to overlapping sprays. It is found, however, that with all fixed nozzles, the maximum quantity of liquid along a radial line under a given head does not fall upon the perimeter of the wetted area. Uniform distribution along any radial line necessitates overlapping the sprays. The overlap required varies with different nozzles and at different heads with the same nozzle. In *d*, Fig. 154, Worcester nozzles having 11/16-in. orifice are operating under a maximum head of 8 ft. They are 14 ft. apart at A, B and C, the point o being 8 ft. from the nozzles. The spray is carried out 10 ft. from each nozzle, as shown by the arcs described about A, B, and C. The extreme overlap beyond the point E, halfway between nozzles, is 3 ft. The figure *abc* is the area of over-

lapping from all 3 nozzles. The overlapping from 3 nozzles on the 8-ft. line *Ao* is very nearly equal to the overlapping from 2 nozzles on the 7-ft. line *AE*.

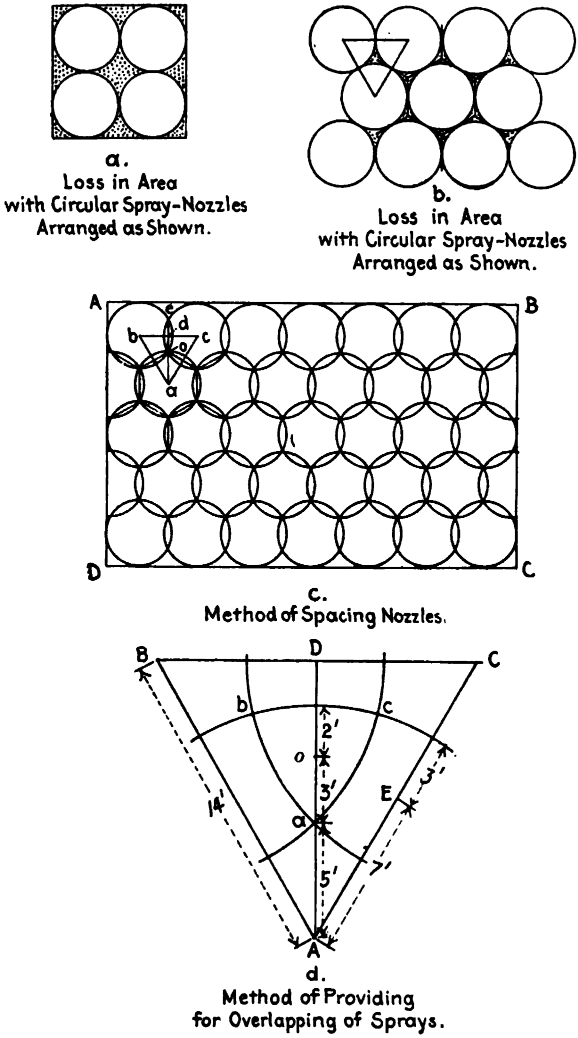


FIG. 154.—Diagrams of nozzle spacing.

There is no overlapping of sprays around the edges of a filter supplied with fixed nozzles, and a smaller quantity of sewage per unit area reaches this portion. Assuming that the capacity of a trickling filter is approxi-

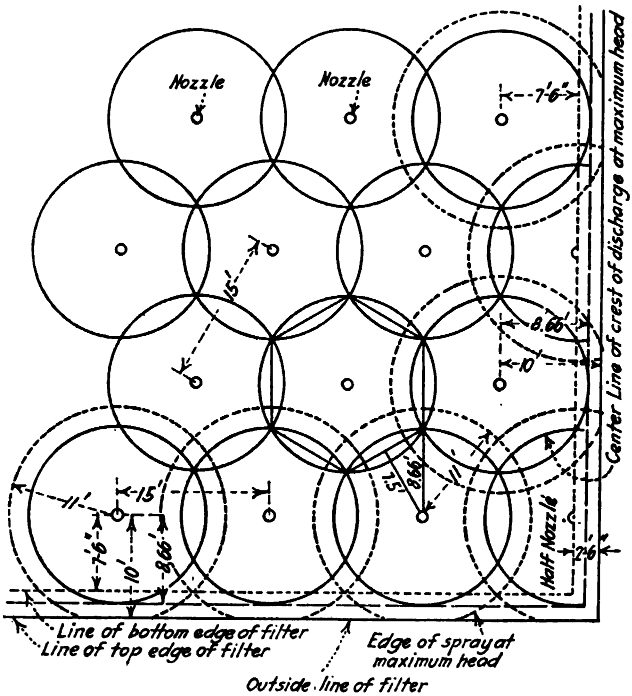


FIG. 155.—Arrangement of nozzles of trickling filter at Fitchburg, Mass.

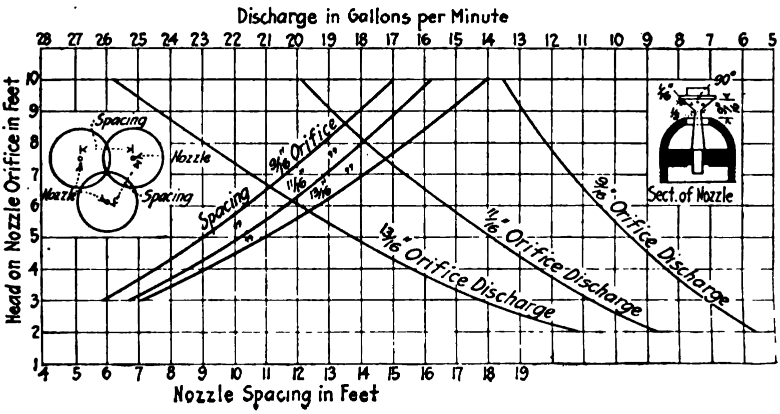


FIG. 156.—Discharge and spacing of Worcester-type nozzles.

mately proportional to the depth, a smaller volume of filtering material may be provided around the edge of a filter, or, in other words, the bottom of the filter may be made smaller in area than the top, thus effecting a considerable saving in filtering material. The provisions made in the Fitchburg designs are illustrated in Fig. 155. A small amount of sewage at the extreme limit of the spray between 10 and 11 ft. from the nozzles, is cared for by building up the side walls sufficiently to retain this spray, as well as that resulting from the effect of the wind. The length and width of the filters at the bottom are 5 ft. less than at the top.

The relation between the head in feet on a Worcester nozzle, the discharge in gallons per minute, and the nozzle spacing in feet is shown in Fig. 156; these curves apply solely to a nozzle of the dimensions indicated on the diagram.

Relative Advantages of Intermittent and Continuous Operation.—It is necessary to vary the head on fixed nozzles in order to secure even distribution over the filter area. It follows, therefore, that no part of the area is in continuous operation, and all parts must be dosed at relatively high rates per unit of area for short intervals of time. Even traveling distributors, which give very even distribution, apply sewage at a rate far exceeding what would be needed by the continuous application of the same daily quantity to the entire filter area.

The period of operation of fixed nozzles, from the time they go into operation at maximum head until they are again in operation at the same head is called the dosing cycle. When a single dosing tank is used, the nozzles are discharging nothing during the filling of the tank and this interval is known as the rest period of the cycle. At a given rate of operation, the rest period will depend upon the size of the dosing tank. The smaller the dosing tank the shorter will be the rest period and the greater the number of dosing cycles. A typical example of the relation of the rest period to the number of cycles at different rates of flow under specific conditions is shown in Fig. 157. Under these conditions there is no rest period above 4,800,000 gal. per acre per day, the nozzles going into continuous operation at that rate. The length of the rest period is limited in cold climates by the danger of freezing the sewage in the exposed portion of the distribution system. A period of 15 or 20 minutes is as much as can safely be allowed under extreme conditions, unless special provision is made for causing a movement of sewage through the distribution system.

If the sewage is allowed to flow into the dosing tank while it is discharging, the sewage flow must be limited to the rate of discharge of the nozzles under minimum head. In many cases this quantity is less than the filter is capable of satisfactorily purifying. Provision should accordingly be made in such cases for storage of the flow during the dis-

charge of the dosing tank or for an additional dosing tank. By the use of double dosing tanks, as at Fitchburg, Chapter XVIII, 1 tank is filling while the other is emptying, thus making possible a much higher rate of operation of the filter under varying head and a shortening of the rest period.

It seems probable, other conditions being equal, that the nearer the application of sewage approaches continuous distribution at the mean rate, the more efficient will be the purification. That the relation of the rest period to the nozzle display period has a marked effect upon the rate of flow of effluent from the filters is illustrated by Fig. 158. (Re-

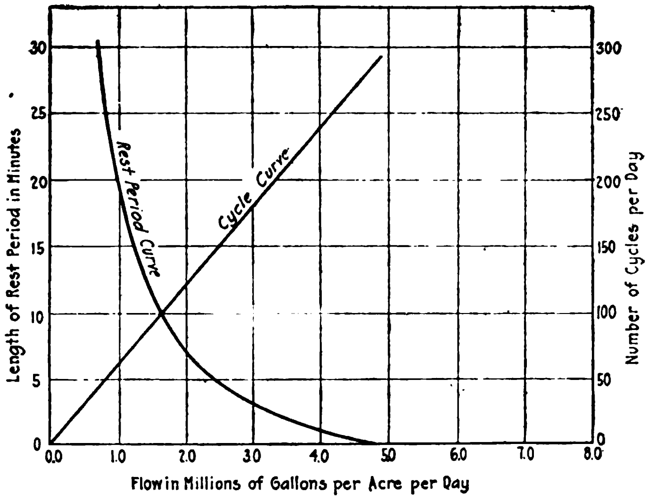


FIG. 157.—Curve of variations in length of rest period and number of cycles per day for various rates of flow of sewage.

Based on performance of Worcester nozzle with $^{13}/_{16}$-in. orifice and 15 ft. spacing, working under heads of 2.04 to 7.43 ft. on a work cycle of 5 minutes.

port Sewage Testing Station, Philadelphia, page 65.) If the flow of effluent from the filter varies markedly, the effluent discharged at the higher rates cannot have received as efficient purification as the same quantity of sewage efficiently distributed so that the effluent emerges from the filter at a constant rate. By shortening the dosing cycle, distribution may be evened up within the filter so that the rate of flow of the effluent is quite constant, which can be accomplished most readily by the use of butterfly valves or their equivalent, described in Chapter XVIII.

There appears to be some difference in opinion as to the value of periods of recuperation of longer duration than the rest periods occasioned by the dosing cycle. Experiments at Columbus indicated that

periods of recuperation were necessary in order to avoid clogging of the filters, while experiments at the Massachusetts Institute of Technology Experiment Station led to the conclusion that recuperation periods were detrimental to the proper working of the bed. The experience of the authors has been that a recuperation period of 24 hours or more is of great assistance at times in aiding trickling filters to unload accumulated solids. Similar resting in rotation of portions of filters affected by organic growths has sometimes proved of marked benefit. The length of the recuperation period will be governed by the extent of the clogging and the results obtained by actual operating tests.

An absolutely uniform distribution of the load upon a trickling filter involves a changing rate of application according to the varying strength of the sewage. Sewage varies greatly in strength during a

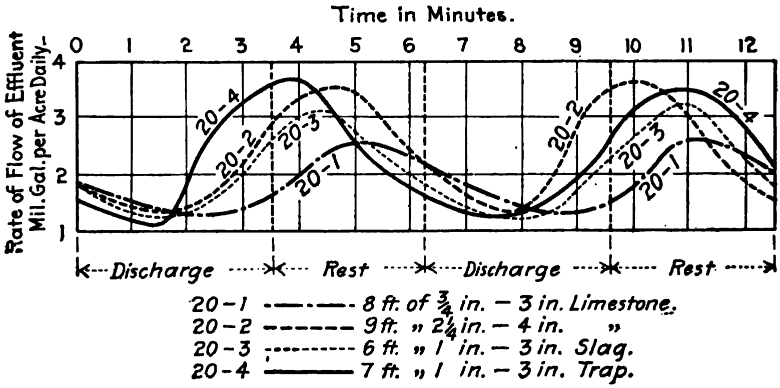


FIG. 158.—Variation in rate of flow of effluent from trickling filter during dosing cycle.

single day, as explained in Chapter V, the maximum strength being often twice the minimum. Moreover, when the strength of the sewage is at its maximum the flow is likely to be double that when the strength is at its minimum. These inequalities in distribution of the load can best be met by resting certain portions of the filter in rotation.

DISTRIBUTION SYSTEMS

Fixed nozzles for distributing sewage to trickling filters involve the use of distribution pipe systems. Distributing mains may be of concrete or vitrified sewer pipe encased in concrete but the smaller sizes are preferably of cast-iron pipe with lead joints. The distribution pipes should be large enough to prevent undue friction and consequent loss in head on the nozzles. Economy of construction requires

the lateral distributors to be relatively short to avoid the use of unnecessarily large pipe.

The distributing laterals may be laid just above the filter floor, as at Columbus, Ohio, Fig. 159; nearer the surface of the filter, as at

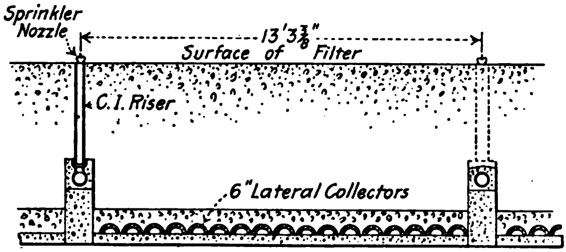


Fig. 159.—Section of floor of trickling filter at Columbus, Ohio.

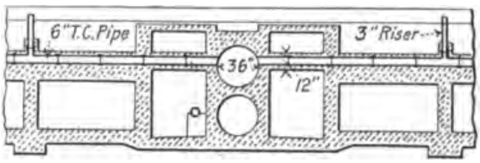


Fig. 160.—Section of portion of trickling filter at Baltimore, Md.

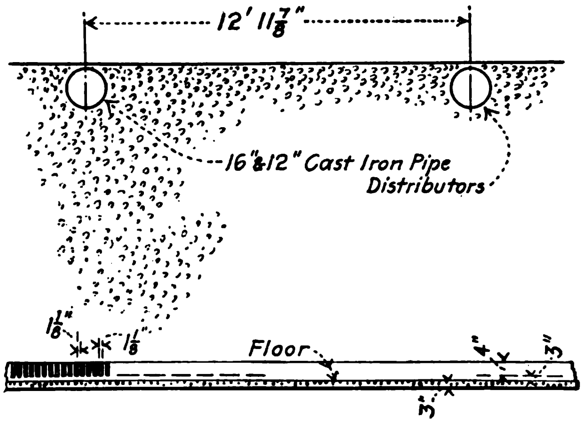


Fig. 161.—Location of lateral distributors in trickling filter at Fitchburg, Mass.

Baltimore, Fig. 160; or at the surface, as in the case of the Fitchburg filter, Fig. 161. The lateral distributors may be allowed to rest upon the filtering material with no other foundation, but if it should become necessary to remove the filtering material for washing or any other

reason, the pipes would then have to be supported or temporarily removed. Where the dash plate or gravity nozzle is used, the distributing troughs or pipes are necessarily supported at an elevation above the surface of the bed. It is advisable in all cases to provide ample facilities for flushing the distribution system, especially as fungus growths are likely to occur.

In certain cases fractures have occurred in lateral distributing pipes owing to excessive pressures. If these pipes are laid at the bottom of the filter, repairs upon them will entail excavation of filtering material, which will add greatly to the cost of the repairs. Placing the laterals at or near the surface makes them more accessible, but large pipe placed at the surface are likely to affect the uniformity of distribution somewhat and clogging is more likely to take place about the pipes than on the remainder of the area. In this case, also, the sewage in the distributing pipes is exposed to low temperatures of a cold climate to a greater extent than when these pipes are below the surface of the bed.

If the lateral distributors are at the surface of the filter, as at Fitchburg, the nozzles may be attached directly to the distributing pipes. If the lateral distributors are placed beneath the surface, riser pipes will be necessary. The most common form of riser is cast-iron pipe jointed with bitumen or some other flexible-joint material. The risers designed by Taylor for Waterbury, Conn., are bitumenized fiber. This material is light and convenient to handle and affords a quick means of attaching the nozzles. Some difficulty was experienced at the experimental filters at Worcester, Mass., in retaining the nozzles in place with these risers, especially after they had become dried out during rest periods. After 2 years of operation with the acid-iron sewage of Worcester, this material was badly disintegrated.

The riser should be large enough to minimize friction, due allowance being made for a considerable deposit or growth on the inside of the pipe. Long risers are more liable to become bent or broken during construction than are shorter ones.

The elevation of the top of the riser above the filter should ordinarily not exceed 6 in., although certain types of nozzle afford better distribution when the nozzle is placed 12 in. or more above the filter surface. The exposed portion of the riser is liable to cause trouble from freezing during the rest periods of the dosing cycle in cold climates. Provision should be made for draining the sewage in the distribution system during long recuperation periods. It is not advisable to place the nozzles below the surface of the filtering material for protection against the frost, because this location may interfere with the spread of the spray, and the nozzles, being less prominent, are more likely to be injured by the workmen or by visitors walking over the beds.

UNDERDRAINAGE SYSTEMS

Trickling filters, built in excavation without masonry floors, are objectionable because the soil may become displaced by water flowing over it, thus causing settlements. Furthermore, the soil may become intermingled with the filtering material and impair the drainage of the filter. Such filters should be built upon floors of concrete or other masonry.

One of the earliest drainage systems consists of a concrete floor upon which is a layer of material composed of pieces preferably not less than 6 in. in diameter, the floor being sloped toward the main collectors. This coarse material affords better drainage than would the finer material of the bed, but clogging is liable to result from the solids unloaded by the filter. The coarse material accomplishes little in the purification of the sewage because of the relatively small bacterial surface per unit of filter volume. In order that the solids may be more readily carried away with the effluent or flushed through the drains, and the gases of decomposition may have a ready means of escape, false floor systems are generally adopted.

The floor system used at Columbus and some other places, Fig. 163, is the least expensive type under some conditions. The concrete slopes toward the main drains and the lateral drains are formed by inverted, half-round, slotted tile placed in contiguous rows practically covering the entire floor. They are bedded in cement to afford an even bearing, thus enabling them to carry the load of stone placed upon them. The pipes are laid with open joints staggered on adjacent rows so that the filter effluent has only a short travel to reach these openings or the openings in the sides of the pipe at the bottom. A layer of coarse filtering material about 4 to 6 in. in diameter is usually spread over the pipes to prevent the finer material from passing through the openings into the drains.

This floor system offers a fairly good opportunity for the passage of solids from the bed into the drains. If the bed is composed of coke, cinders, or other material liable to disintegrate, the valleys between the pipes will probably become gradually filled with fine material which may clog the drains. A disadvantage of this floor is that the flow of effluent is distributed over the floor beneath the tile so that a relatively low velocity is afforded for carrying away the solids. Furthermore, it is not readily flushed because the water spreads over the bottom, rendering the stream ineffective.

Fig. 163 also shows the floor system designed for Waterbury, Conn. The concrete is flat, sloping toward the main drains. A special block of vitrified clay or reinforced concrete is set on the floor, furnishing a false bottom over the entire area. The drainage area is much greater

than in the case of the Columbus type and the effluent flow is confined to the drains. The effluent is allowed to spread out over a very large part of the floor, however, so that the self-cleansing qualities and facilities for flushing are not ideal.

The Baltimore floor system, Fig. 163, consists of a series of grooves and ridges in the concrete bottom. The grooves are covered with slotted vitrified clay or concrete slabs bedded in cement, concentrating the flow of effluent in narrow channels. The floor of the filter slopes toward the main drains and the inverts of the lateral drains are parallel with the floor, which necessitates a varying depth of stone in accord-

FIG. 162.—Constructing trickling filter floor, Fitchburg, Mass.

ance with the slope of the floor. While the practical objections to such a variation in depth of filter may be questionable, it is theoretically an advantage to build the bed of uniform depth. With this type of floor system it is feasible, although possibly somewhat more expensive, to lay the floor level and give the necessary grade to the drains. In this case the filtering medium will be of uniform depth.

A type of floor designed by the authors and used at Fitchburg, Mass., is shown in Fig. 163. The lateral drains in the concrete are covered with narrow cement beams, affording a very large percentage of drainage area as well as a concentrated flow of effluent. A photograph of this floor during construction is reproduced in Fig. 162. Cobble stones,

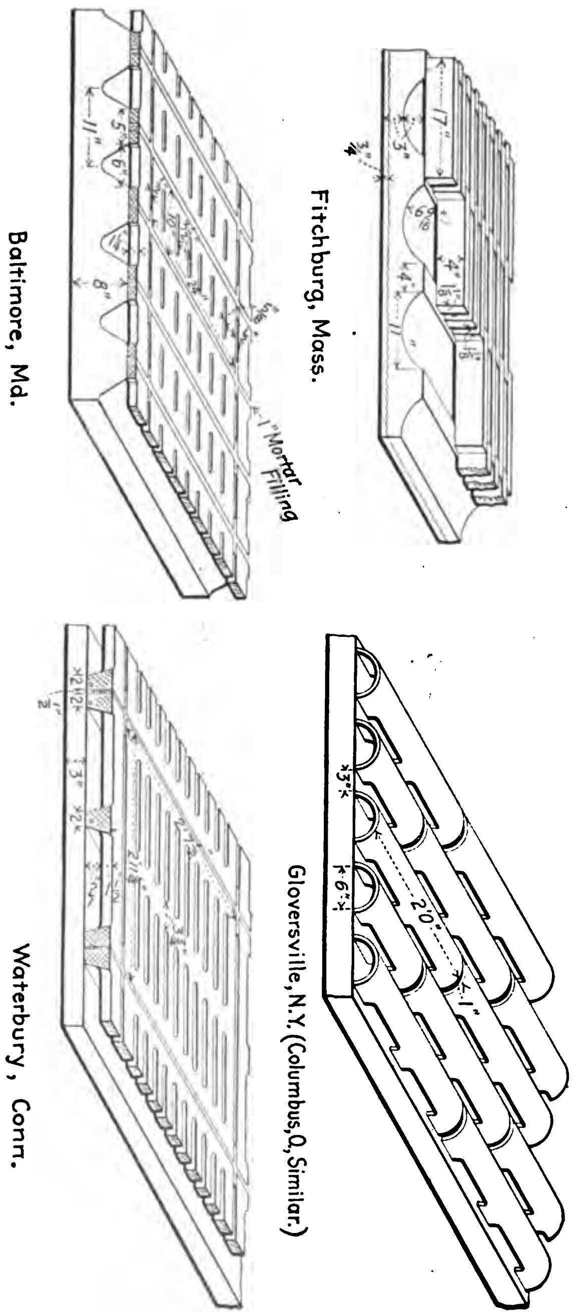


FIG. 163.—Types of floors of American trickling filters.

which were abundant on the site, were placed by hand over the openings before placing the crushed stone on the bed.

A number of the important elements governing the selection of a type of floor system are given in Table 133. The thickness of the floor usually has an important bearing upon the cost of the filter. The percentage of open area in the false floor for drainage and the proportion of free space for carrying away the effluent and gases of decom-

FIG. 164.—Flushing gallery of trickling filter, Fitchburg, Mass.

position should be as large as can be provided at reasonable cost. The strength of the false floor should be ample for the load to be placed upon it.

TABLE 133.—ELEMENTS OF UNDERDRAIN FLOOR SYSTEMS SHOWN IN FIG. 163

City	Percentage of surface which is drainage area	Percentage of volume which is air space	Maximum thickness of floor, inches
Baltimore, Md....	7.6	13.7	9¼
Columbus, Ohio...	11.5	45.4	7⅝
Fitchburg, Mass..	40.3	33.5	10
Waterbury, Conn..	22.2	30.0	6½

The possibility of clogging the drains with solids unloaded from the filter or by organic growths in them necessitates ample provision for flushing. In some cases the upper ends of the laterals are carried through the filter wall, thus affording an opportunity for flushing. In other cases, flushing galleries have been provided, Fig. 164. The

size of the lateral drains should be sufficient to carry away the effluent promptly, to make flushing possible, and to afford space for the circulation of air.

Economy of construction usually requires the lateral drains of the floor system to be relatively small and short, so that it is necessary in most cases to provide larger main drains into which the laterals may empty. In large plants it may be advisable to have these main drains discharge into still larger collectors. The arrangement of lateral and main drains in an early type of floor system at Batavia, N. Y., is shown in Fig. 165.

The main drains may be in the form of channels covered with slabs, or they may be circular or semicircular in shape. An open channel

Fig. 165.—Floor system of trickling filter, Batavia, N. Y.

around the outside of the filter serving as a main drain affords an excellent means of ventilation, for the ends of the lateral collectors are exposed to the air and the wind undoubtedly induces air currents through them (Fig. 133). It is important to give grades to all parts of the drainage system which will prevent the effluent from backing up in the ducts and channels.

VENTILATION

Inasmuch as the proper action of the filter depends on an abundance of oxygen within the bed throughout its depth at all times, adequate ventilation is essential. Downward ventilation is secured by the natural flow of sewage through the bed. Any interference with the

Fig. 166.—Ventilating chimneys of trickling filter, Salford, England.

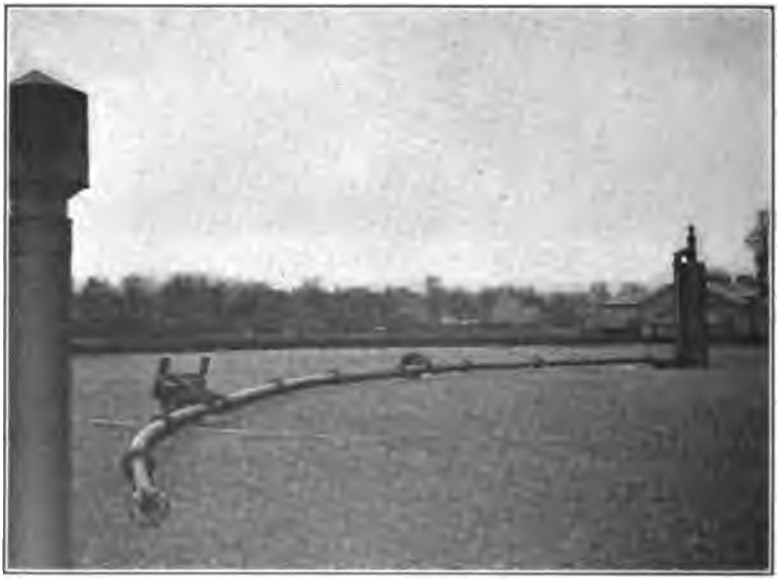


Fig. 167.—Ventilator at circumference of trickling filter at Worcester, England.

free passage of water and air, due to the accumulation of clogging matters or to insufficient drainage capacity, causes a decrease in the efficiency of the filter.

FIG. 168.—Trickling filter with ventilators, Atlanta, Ga.

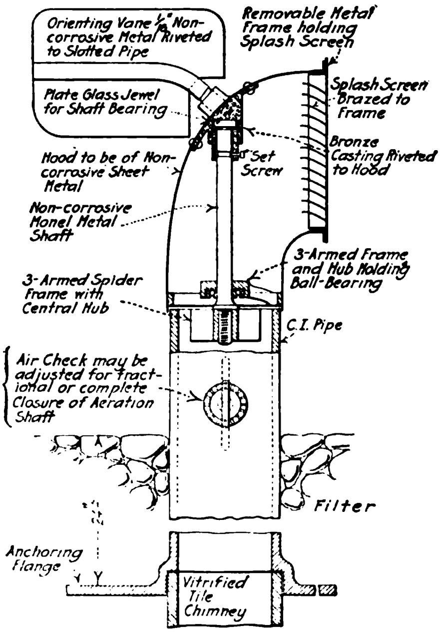


FIG. 169.—Ventilator proposed for trickling filter at Waterbury, Conn.

Some filters have walls of large stones, Fig. 131, with openings for air to enter the bed laterally. In other filters openings are left in brick walls, Fig. 133, for the same purpose. While this may prove beneficial for

small filters, its value for large units is doubtful. Any advantage gained during warm weather may be more than offset by decreased bacterial activity in cold weather, owing to lower temperatures produced in the filter.

At Gloversville, manholes were provided at the ends of the flushing galleries to aid in ventilating the filters. In several instances, vent pipes connected with the floor system and extending above the filter surface have been provided at regular intervals. At Philadelphia, Pa., vitrified pipe risers were placed 10.8 ft. on centers over the main underdrains. The system at Salford, England, Fig. 166, consists of brick chimneys placed over the main drains.

Investigators in England and the United States have experimented with forced draft, which is rarely attempted at the present time. The same effect may be secured in a measure by employing a series of vertical pipes connected with the underdrains and provided at the upper ends with cowls, thus making use of the wind for aeration. The type of ventilator used at Worcester, England, around the edge of the circular filter is shown in Fig. 167. Cowl ventilators in use at Atlanta, Ga., Fig. 168, consist of 8-in. cast-iron pipe risers with revolving cowls composed of a copper hood and wind vane. They are placed over the main underdrains in the proportion of about 72 ventilators to the acre. The illustration shows that they do not always revolve with the wind. The details of a cowl ventilator designed by Taylor for Waterbury, Conn., are given in Fig. 169 (*Engineering News*, August 19, 1909). While special ventilators may be of value, they are by no means essential under ordinary circumstances where liberal provision is made for free drainage. It would be unwise to curtail the drainage facilities where ventilators are contemplated.

WALLS

Where filters are in excavation, the soil should not mingle with the filtering material. This can be assured by sheeting the banks with planks which will later keep the soil from the filtering medium. Such sheeting can rarely be kept tight and there is liability of fine soil sifting through the cracks into the filter. Moreover, the life of lumber is relatively short.

Some form of masonry is best for filter walls, which need not necessarily be made strong enough to act as retaining walls to support either the natural banks or the filtering material, although if such is the case care should be taken not to place excessive loads upon them during construction and to take proper precautions when the filtering material is removed for washing. If the ground around the filter is higher than the surface of the bed, care should be taken to prevent washing

of the soil into the filtering material by sewage spray or by storm water. This may be accomplished by carrying the walls up above the ground elevation or by providing drains to carry away the surface wash.

Types of wall used at Columbus, Ohio, are shown in Fig. 170. The wall of the trickling filters at Gloversville, N. Y., Fig. 171, is buttressed and reinforced to support the roof system. At Fitchburg, Mass., ribbed metal lath plastered on both sides was used as a curtain wall between the natural soil and the filtering material. In this case, the wall is inclined from the bottom outwardly as explained on page 590.

If trickling filters are constructed above the surface of the ground, the filtering material may be allowed to assume its natural angle of repose. This method of construction, however, involves the use of considerable filtering material which is ineffective in the treatment of the sewage, and the construction of a larger floor system than is required

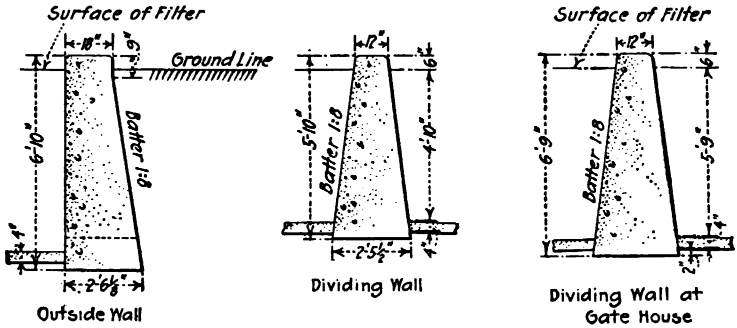


FIG. 170.—Walls of trickling filter, Columbus, Ohio.

to collect the effluent. For these reasons such construction is not economical in most cases where crushed stone is used for filtering, but where an inexpensive material is available, it may be the cheapest construction.

If the filtering material is not sloped as above described some form of retaining wall will be necessary. Such walls have been constructed of tile, brick, concrete, etc. The brick wall, Fig. 133, is commonly seen in England. This type is calculated to facilitate aeration within the bed. To accomplish the same purpose tile have sometimes been built into solid masonry walls.

FILTER UNITS

The shape of the filters will be controlled in part by the topography of their site and to a large extent by the distribution scheme used. The size of circular filters with mechanical distributors is limited by the economical size of the distributing device, and is influenced by the fact

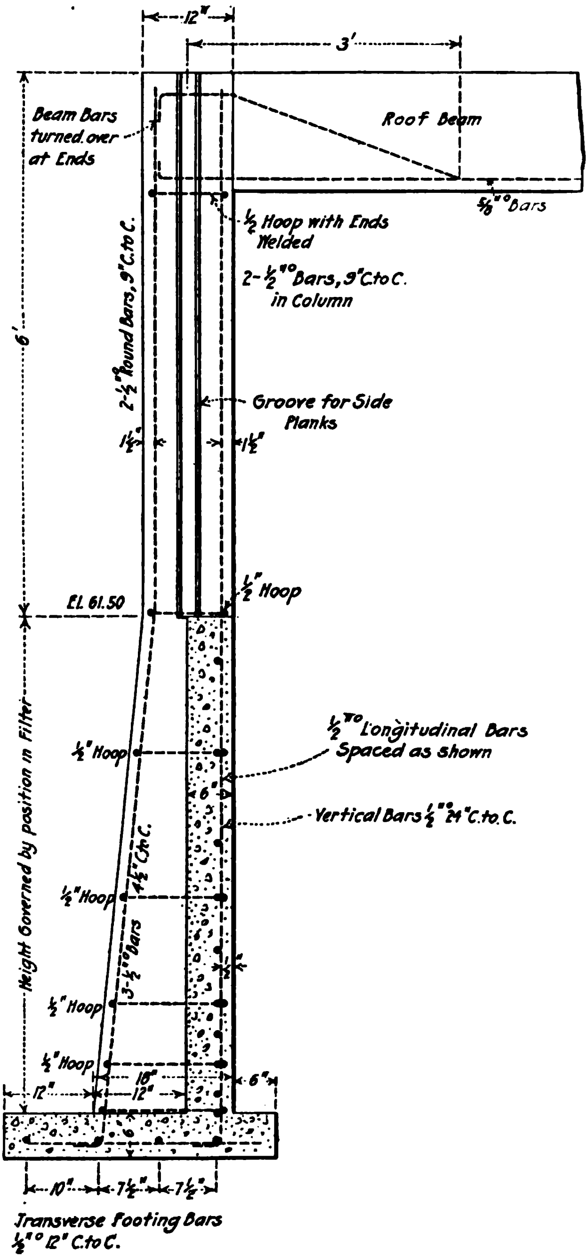


FIG. 171.—Wall and roof beam of trickling filter at Gloversville, N. Y.

that it is less expensive to build a few large beds than many small ones. For filters using mechanical distributors more than 1 unit should be provided, so that in case repairs become necessary the entire plant need not be shut down. The Royal Commission on Sewage Disposal recommended not less than 3 three units for each plant. (Fifth Report, page 94.) There is a decided advantage in having small units so that por-

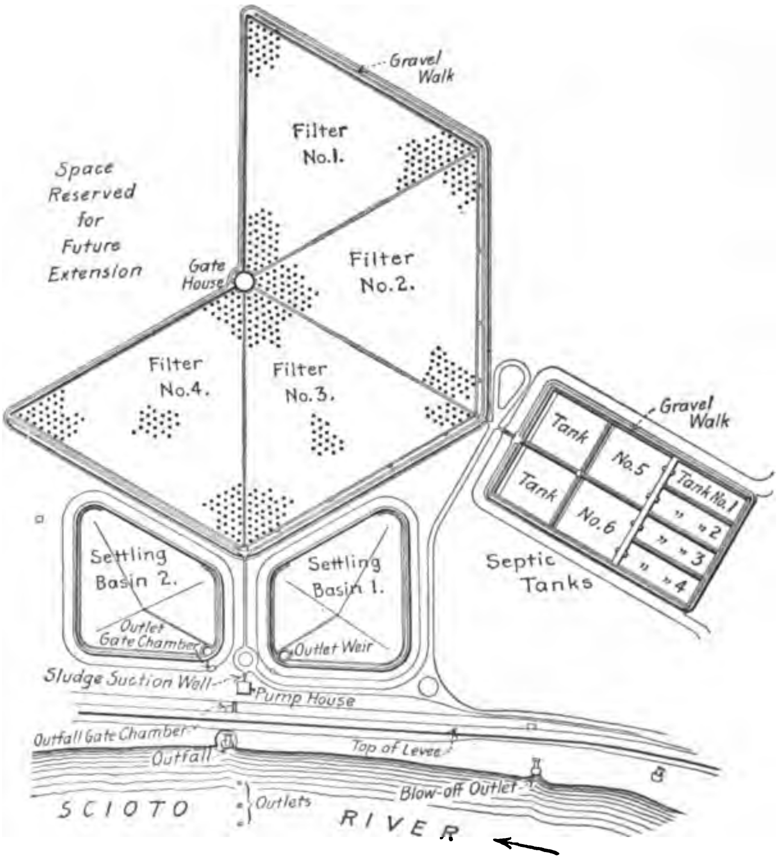


FIG. 172.—Arrangement of sewage treatment works at Columbus, Ohio.

tions of the filter may be rested at intervals. A filter fed by fixed nozzles may be divided into as small units as desired.

At Columbus, Ohio, the 10 acres of trickling filter are divided into 6 units, each an equilateral triangle and all arranged about a central point (Fig. 172). This plan is advantageous for distribution but not for some forms of underdrainage. It has proved unsatisfactory to have

the administration building in the center of the plant owing to obnoxious odors and flies. At Baltimore, there are 4 separately operated units for 12 acres of filter, while at Philadelphia 5 units have been provided for a single acre. At Fitchburg, each lateral distributor is provided with a valve, thus making it possible to cut out any one or more of 17 lateral distributors provided for a 2-acre bed.

PRELIMINARY TREATMENT REQUIRED

Some form of preparatory treatment will ordinarily be required to prevent filter clogging. How far such treatment should be carried will depend upon the size of filtering medium and the economy of construction and operation of the treatment plant. Washing clogged filtering material should be avoided if possible. Grit chambers and coarse screens alone will rarely be sufficient. Fine screens may be adequate in certain instances, although the general opinion (in 1915) is that sedimentation tanks remove suspended solids more economically and efficiently. They should be designed and operated so as to take out all the settling solids it is practicable to remove.

A disadvantage of the septic tank for preliminary treatment is the foul odors often given off by the tank effluent. Spraying the effluent on trickling filters is particularly favorable to the dissemination of the odors. Further, there is a general belief that sewage can be purified by bacterial agencies more advantageously when fresh than when in a septic state.

At Worcester, Mass., chemical precipitation was employed as a preparatory treatment for a large experimental trickling filter for over 1 year, and the results were compared with those obtained when using plain sedimentation. The conclusions reached were as follows:

"These results indicate that about 30 per cent. more of the chemical effluent than of the corresponding settled sewage may be applied to sprinkler filters with like results. The results obtained by the treatment of chemical effluent on the 5-ft. filter were decidedly inferior to those obtained by the treatment of settled sewage on the 7.5-ft. filter of the same material operating at the same rate.

"The amount of dissolved organic matter is quite as high in chemical effluent as in settled sewage, but the chemical effluent has advantages among which may be mentioned, a less amount of suspended matter to be oxidized within the filter, an alkaline medium for nitrification instead of an acid medium, and the absence of a considerable amount of ferrous iron sulphate originally present in the sewage which takes all of the oxygen required for its oxidation before nitrification can take place.

"When the cost of chemical precipitation is taken into consideration, it is obvious that the increase in rate made possible by this treatment is more than offset by the extra cost." (Report of Supt. of Sewers, 1911, page 500.)

The Royal Commission on Sewage Disposal reached the following conclusion:

"In the absence of special circumstances favoring a particular plan, it would appear that there is very little difference in annual cost between the various methods of tank treatment followed by filtration through percolating filters, assuming that the kind of filter adopted in each case is that which is best adapted to the particular tank treatment provided." (Fifth Report, page 43.)

Clogging of Nozzles.—Some types of nozzles require very complete removal of suspended matter and even with careful preliminary treatment there is likely to be more or less trouble from their clogging.

At Birmingham, after thorough preliminary sedimentation of the sewage, nozzles having an orifice in the shape of an annular ring $5/64$ in. wide became clogged badly with fatty substances. Strainers installed between the sedimentation tanks and filters largely overcame this difficulty and reduced the daily cost of maintenance per acre of filters from $1.09 to $0.38.

At Columbus, Ohio, much trouble from clogging of nozzles has been reported during times when the tanks are in a septic condition, in spite of the fact that nozzles having a free orifice $9/16$ in. in diameter are in use.

At Reading, Pa., 18 per cent. of the nozzles were cleaned daily during 1912. The orifice of these nozzles is an annular ring $5/16$ in. wide. The tank effluent filtered is in a septic condition much of the time.

At experimental trickling filters at Worcester, Mass., considerable trouble was experienced with fatty substances clogging nozzles of the Columbus type, having $3/8$-in. orifices, when treating the effluent from chemical precipitation. A wire screen of $1/4$-in. mesh, placed about the end of the inlet pipe, largely overcame this difficulty, but usually needed to be cleaned once a day. During severe winter weather a coating of carbonate of lime was formed upon everything with which the chemical effluent came into contact, thus reducing the size of the distributing pipe and nozzle orifices, which increased the clogging of the nozzles and their consequent freezing. Much more trouble was experienced with nozzle clogging when treating septic tank effluent. When treating the effluent from Imhoff tanks, however, practically no clogging with sewage matters resulted, even when using the Worcester nozzle having an orifice in the form of an annular ring $1/4$ in. wide. The maximum head on the nozzle orifice in this case was 8 ft., whereas it was 5 ft. in the other cases, and doubtless less trouble was experienced from nozzle clogging because of the increased maximum head upon the nozzle orifices.

ORGANIC GROWTHS, WORMS AND INSECTS

Even though trickling filters be carefully designed to avoid clogging with suspended solids, trouble may result from organic growths. Ex-

cessive growths in English filters occur mainly in the winter and early spring and apparently are not due to the kind of preliminary treatment, although the Royal Commission on Sewage Disposal stated (Fifth Report, page 97) that the growths were most abundant on filters receiving chemical precipitation effluent, but may have been due then to the large volumes treated. At Dorking, different portions of the clogged surface were treated with powdered quicklime, bleaching powder, copper sulphate, ferrous sulphate, caustic soda, and sulphuric acid, respectively. The quicklime and bleaching powder were applied in a thin layer. The other substances were applied in the form of a 20 per cent. solution at the rate of 1 liter to 1.6 sq. yd. Only caustic soda was effective, but in this case the growth was almost entirely destroyed and appeared in large quantities in the effluent when the filter was again put into operation. The efficiency of the filters was not impaired.

At Waterbury, Conn., a "slimy vegetable growth" appeared upon the surface of experimental trickling filters soon after they went into operation, and attained a thickness of nearly ½ in. (*Engineering News*, November 1, 1906). The color varied from a salmon pink to a jet black. Isolated cultures showed a "sausage chain" of small oval bodies closely resembling monillia. The growth was satisfactorily eliminated by resting the bed 2 days out of 7, although it was more quickly removed, after a day's rest, by breaking up the surface layer with a pick and washing with a hose. In this case the sewage was fresh and alkaline.

An experimental trickling filter at Andover, Mass., in 1906, became clogged with a "growth of an amorphous jelly-like mass of bacteria." The applied sewage was treated with a strong solution of copper sulphate, which caused the growth to disappear quickly and did not interfere with the efficiency of the filter. (Report Mass. St. Bd. Health, 1908, page 382.)

At the sewage testing station at Philadelphia, a luxurious pinkish growth, similar to that at Waterbury, occurred upon the surface of several filters, and was destroyed by resting the beds in warm dry weather. When the sprinkling failed to flush out the accumulated matter, washing with a fire hose was effective without injury to the bacterial efficiency of the filter. Water was used at the rate of 115,000 gal. per acre, and 2 or 3 men per acre were employed in the washing. The application through the nozzles, of a strong solution of bleach in the proportion of 2 tons to the acre was found to be very efficient and much more economical than the application of dry bleach. In either case considerable available chlorine was carried away in the effluent and the growth soon appeared again. The most satisfactory method of applying the bleach was by a continuous disinfection of the influent, which aided in the purification and in no way interfered with the bacterial activity in the filter (Report on Sewage Testing Sta-

tion, Philadelphia, 1911). This experience coincides with the conclusion of the Massachusetts State Board of Health that sewage may be sterilized by chlorine added as bleaching powder, without affecting the subsequent purification by sand or trickling filters (1908 Report, page 363).

At Worcester, Mass., growths appeared annually on the experimental trickling filters upon the advent of cold weather and disappeared during the summer. Two species have caused considerable trouble: one a mould forming pink or brown gelatinous colonies resembling leptothrix under the microscope, the other a white or gray filamentous growth resembling leptomitus. Permanent clogging of these filters has been prevented by resting them periodically for a day or more and loosening the surface with a pick. By this means the growth is partially dried and then washed away. Possibly the same result could be obtained practically and economically on a large scale, by the use of a spike-tooth harrow. Satisfactory recuperation was afforded by shutting down one-seventh of the area for 1 day each week.

A rubber-like growth occurred on the Birmingham, England, filters during cold weather when fresh sewage was being treated, but when septic tank effluent was applied, no marked growth occurred. This led Watson to suggest the application of septic sewage during winter. (Birmingham, Tame and Rea District Works, 1912, page 36.) It was possible to keep down this growth by applying the sewage intermittently.

Worms.—After a trickling filter has been in operation a short time in warm weather, large numbers of worms of different species are often found in the deposit in the upper portion of the filtering material. A species similar to the earthworm is often very abundant, as at the Worcester experimental filters.

"During warm weather these were observed in the top stone of the beds in masses of minute worms. Deeper down they were much larger in size, many of them being several inches long. They appeared to thrive equally well throughout the winter season. When the solid matter within the filter became oxidized to such an extent that it sloughed off from the stone, multitudes of the worms were flushed out with the so-called humus. At times they could be taken out of the distributing troughs of the humus sludge tanks by the handful. They were always present in the humus sludge in large quantities, and it is believed that they were responsible for the disagreeable fishy odor characteristic of this sludge." ("Experimental Treatment of Sewage, Worcester," Gault, 1912, page 51.)

It seems probable that a vast amount of work in the transformation of organic matter into more stable forms is accomplished by the worms and insects within the filter. It is apparent, however, that a certain amount of bacterial oxidation must take place before such organisms as the angleworm can live in sewage deposits. Undoubtedly the ac-

tion of the worms upon the solid matter within a trickling filter changes it into a condition to be more easily unloaded from the bed.

Flies.—A small, gray moth-fly (Psychoda) usually becomes very noticeable about trickling filters in early spring and increases in number as the weather becomes warmer. With the advent of cold weather they seek refuge in the body of the filter and many die, but some have been found more or less dormant throughout the winter. They appeared on the experimental trickling filters at Worcester within a month after the beds went into operation.

These flies cause much annoyance to the filter attendants by getting into their eyes, nostrils and ears. Apparently they do not ordinarily thrive apart from the filter but they may be carried considerable distances on the clothing or by the wind. A tannery about 200 ft. from the Gloversville filters experienced considerable inconvenience from these flies getting upon the skins while they were being tanned. At Worcester, swarms flew at times about the buildings 100 yd. from the filters. They were also found in large numbers upon old sludge beds more than 1000 ft. from the filters. Although it was not proved that they originated in the trickling filters, this variety of fly had never been observed on these sludge beds in previous years.

Worms and the larvæ of flies may appear in such amounts as to cause filter clogging. A trickling filter at Andover, Mass., became clogged in the upper portion with fly larvæ. Copper sulphate was mixed with the applied sewage in the proportion of 830 lb. per 1,000,000 gal. on four occasions and bleaching powder in the same proportion on another occasion. Copper sulphate was somewhat more effective than bleach, but neither was entirely satisfactory in relieving the clogging. Caustic soda solution applied to the surface in the proportion of 1000 lb. of caustic soda per acre was much more effective. Digging over the surface layer and resting was sufficient to entirely remove clogging.

Many species of insects other than flies have been observed in trickling filters, particularly spiders, which evidently feed upon the young flies. The destruction of insects by disinfectants is taken up in Chapter XIX.

SEDIMENTATION OF TRICKLING FILTER EFFLUENTS

If the trickling filter properly performs its function the sewage matters which accumulate in the filtering material will not be permanently retained in the bed, but will be unloaded and carried away in the effluent as explained on page 229. As much suspended matter may be expected in the effluent as in the influent; indeed, colloidal matter is often precipitated within the filter so that the effluent averages higher in suspended matter than the influent. This difference may be further augmented by bits of organic growths, worms, etc., carried through the filter.

TABLE 134.—RELATIVE QUANTITIES OF SUSPENDED MATTER IN INFLUENT AND EFFLUENT OF TRICKLING FILTERS AT READING, PA.

(Parts per 1,000,000; Report of Edmund B. Ulrich, City Engineer, 1912)

1912	Filter influent	Filter effluent No. 1	Filter effluent No. 2	Filter effluent No. 4
January	142	62	45	36
February	151	70	57	48
March	128	61	76	58
April	114	56	96	79
May	119	38	57	62
June	143	54	60	51
July	158	71	47	53
August	109	23	23	28
September	103	35	28	33
October	128	43	58	65
November	130	53	68	70
December	138	50	58	60
Averages	130	51	56	54

TABLE 135.—RELATIVE QUANTITIES OF SUSPENDED MATTER IN INFLUENT AND EFFLUENT OF TRICKLING FILTERS AT COLUMBUS, OHIO

(Parts per 1,000,000; Report, Division of Sewage Disposal, 1911)

1911	Influent Maximum	Influent Minimum	Influent Mean	Effluent Maximum	Effluent Minimum	Effluent Mean
January	....	....	....	....	....	....
February	....	....	....	....	....	....
March	149	43	77	106	31	52
April	235	83	122	124	53	85
May	156	65	108	156	59	102
June	306	59	133	227	75	124
July	330	88	125	218	70	122
August	180	68	107	225	66	113
September	105	40	76	127	10	67
October	94	61	78	42	11	25
November	....	....	....	....	....	....
December	....	....	....	....	....	....
Averages	330	40	99	227	10	96

TABLE 136.—RELATIVE QUANTITIES OF SUSPENDED MATTER IN INFLUENT AND EFFLUENT OF TRICKLING FILTERS AT BIRMINGHAM, ENGLAND

(Parts per 1,000,000; "Works of Birmingham, Tame & Rea District Board," Watson, 1912)

	1908		1909		1910		1911	
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
January	120	68	141	105	72	87	....	88
February	85	110	111	111	55	85	69	115
March	....	145	156	130	107	118	57	125
April	....	210	110	185	125	174	91	182
May	202	255	116	171	86	178	159	225
June	102	197	123	234	157	150	134	163
July	128	136	99	129	114	100	89	76
August	102	161	74	73	88	105	63	82
September	100	121	87	117	140	95	....	88
October	136	149	43	85	87	126	78	79
November	162	125	59	68	108	82	75	72
December	123	129	67	87	114	79	75	70
Average	126	150	99	125	104	115	89	114

The 1912 records at Reading, Pa., shown in Table 134, indicate a net storage of solids within all filters throughout the year. Under such circumstances special precautions must be taken to prevent filter clogging and the possible ultimate necessity of washing the filtering material. The results at Columbus, Ohio, Table 135, show a marked seasonal variation in the quantity of suspended matter in the effluent of the trickling filters, but the average results for the year indicate no material net storage of solids within the filter. The results at Birmingham, England, Table 136, also show more suspended matter in the effluents during the summer, as do the annual[1] averages.

In appearance, the suspended matter of trickling filter effluents is quite different from that in sewage. It is of a granular and gelatinous nature instead of the slimy, mucilaginous character of sewage sediment. The increase in stability of the suspended matter in trickling filter effluent over that in the applied sewage was shown by an experiment at the Lawrence Experiment Station in which 0.2 gram of each sediment was mixed with 4000 cc. of river water saturated with oxygen. (1908 Report, Mass. St. Bd. Health, page 376.) The results are given as follows:

	Dissolved oxygen at end of 5 days; percentage of saturation
River water	90.0
Trickling filter 135 sediment and river water	100.0
Trickling filter 136 sediment and river water	76.0
Sewage sediment and river water	1.5

[1] Attributed by Watson to colloids intercepted in filters and washed out later.

It seems to be clearly established that the deposit from trickling filters is much more stable than that from sewage; yet it appears that the effluent may carry, at times of unloading, such large quantities of this matter that the water is rendered putrescible thereby. Sludge resulting from the sedimentation of trickling filter effluents has usually been found to be putrescible. In some cases it has been rendered offensive by the presence of many decaying worms or by particles of organic growths.

In the great majority of trickling filter installations it will be necessary to remove suspended matter from the effluent by tanks, strainers or filters. Much of the suspended matter lends itself readily to sedimentation, but there is considerable fine flakey matter which does not settle readily. Quiescent sedimentation is required to remove a large percentage of this matter, and for complete removal, strainers or filters must be used. In the latter case it will probably be economical to pass the trickling filter effluent through sedimentation tanks first. Intermittent sand filters will not only remove this suspended matter but will afford further bacterial purification and may be operated at comparatively high rates, as at Gloversville, N. Y.

At Columbus, Ohio, shallow horizontal-flow tanks holding about one-third of the daily dry-weather flow are capable of removing about 50 per cent. of the suspended matters in the trickling filter effluent, leaving about 50 parts per 1,000,000 in the final effluent. (1910 Report, Division for Sewage Disposal.) At Reading, Pa., shallow tanks with side walls sloping outward at an angle of 45 deg., holding 2 to 3 hours' average flow, are used. During 1912 about 50 per cent. of the suspended matter in the effluent was removed, leaving an average of approximately 25 parts per 1,000,000 in the final effluent.

At Birmingham, Watson found that sedimentation in shallow tanks was not satisfactory because the surface velocity carried a large portion of the suspended solids away in the effluent. Tanks with hopper-shaped bottoms of the Dortmund type, Fig. 87, have proved very efficient at Birmingham. Vertical-flow tanks have been used for this purpose at Gloversville, N. Y., and Fitchburg, Mass., on the advice of the authors.

Loss in Head.—The loss of head in a number of sewage treatment plants where trickling filters are used is given in Table 137. The Columbus data were given by Gregory in *Trans. Am. Soc. C. E.*, vol. lxvii, 1910, page 305; the Fitchburg and Gloversville data are from the authors' plans; the Schenectady data from Fuller's contract drawings and specifications, and the Washington data from Pratt's article in *Engineering News*, July 16, 1908.

TABLE 137.—LOSS OF HEAD IN FEET IN SEWAGE TREATMENT PLANTS WITH TRICKLING FILTERS

	Elevation, feet	Head lost, feet
Columbus, Ohio:		
High-water line, primary septic tanks	31.34	....
Low-water line, primary septic tanks	28.24	3.10
Hydraulic grade at sprinkling nozzles	25.15	3.09
Surface of filter	20.15	5.00
Hydraulic grade at lower end of main collector.	13.77	6.38
Hydraulic grade, effluent well in gate-house	11.96	1.81
Water line in sedimentation basins	11.00	0.96
Mean low water in river	6.00	5.00
Total	....	25.34
Fitchburg, Mass.:		
Flow line, Imhoff tank	385.4	....
Flow line, dosing tank	384.0	1.4
Surface of trickling filters	375.0	9.0
Invert of main underdrain	360.7	14.3
Flow line, in sedimentation tank	360.0	0.7
Invert of outlet channel at river[1]	343.3	16.7
Total	....	42.1
Gloversville, N. Y.:		
Flow line, primary sedimentation tanks	67.0	....
Surface of trickling filters	61.5	5.5
Invert of main underdrain	55.0	6.5
Water line in secondary sedimentation tanks.	55.0	....
Surface of sand filters	51.0	4.0
Invert of outlet at creek	45.6	5.4
Total	....	21.4
Schenectady, N. Y.:		
Flow line, primary sedimentation tanks	13.75	....
Surface of filter	9.00	4.75
Flow line, secondary sedimentation tanks	3.67	5.33
Invert of outlet at sec. sed. tanks	0.05	3.62
Total	....	13.70
Washington, Pa.:		
High-water line, septic tanks	997.0	....
Low-water line, septic tanks	995.0	2.0
Surface of filter	991.0	4.0
Invert of main underdrain	980.5	10.5
Total	....	16.50

[1] This large loss of head due to topography.

SOME ASPECTS OF OPERATION

Trickling Filter Plant Odors.—The spraying and aerating of sewage is favorable to the release of gases and the dissemination of odors. The odor of fresh sewage is soapy or like that of raw turnip, but after putrefaction has set in, the odor may become very offensive. It is highly important, therefore, to deliver sewage to trickling filter plants as fresh as possible and to carry out the preliminary treatment promptly, all sludge deposits with which the sewage is in contact being removed at frequent intervals.

At Reading, where the sewage is applied fairly fresh to the trickling filters, it is said that odors are not noticeable 100 yd. from the plant. At Columbus, on the other hand, where the sewage is in a more or less septic condition, it has been claimed that at times foul odors from the plant have been observed half a mile away, although Fuller states that they are not normally noticeable more than 300 yd. away. ("Sewage Disposal," page 718.)

In some cases it has been found practicable to minimize foul odors by the use of lime, bleaching powder or other chemicals, but such treatment adds materially to the cost of operation. Planting trees about the filter may assist in preventing the dissemination of odors by the wind. Poplars are advantageous because of their rapid growth, but evergreens should be planted at the same time on account of the short life of the poplars. The grounds about the sewage treatment plant at Pennypack Creek, in Philadelphia, consisting of Imhoff tanks and 1 acre of sprinkling filters, are being developed as a city park. At Mt. Vernon, N. Y., the filter area of 1.2 acres in the midst of a residential section is housed, and the air above the filters is forced through purifying towers. (*Engineering News*, April 29, 1909.)

Efficiency of Trickling Filters.—In Table 138 are shown average results obtained at Birmingham, Columbus, Reading and Worcester. The results give only the average efficiency. The effluent during a single day may vary greatly on account of the variation in load placed upon the filter. The effluent produced from the strongest day sewage may be putrescible, while the average effluent for the day is stable. There is usually a seasonal variation in the purification effected, like that shown in Fig. 173, illustrating the seasonal variation in the effluent of 10-ft. trickling filters operated at a rate of 2,000,000 gal. per acre daily, at Worcester.

Conditions other than temperature affect the efficiency, such as the dilution of the sewage with surface and ground water, particularly during the spring months, and the presence at times of organic growths on the surface which tend to obstruct the passage of sewage and air. The deterioration of the effluent in winter may be explained partly by

a decrease in the available area due to ice. At Worcester, it was estimated that when the temperature went to –10°F., less than 50 per

TABLE 138.—EFFICIENCY OF TRICKLING FILTERS AND SECONDARY SEDIMENTATION TANKS

	Birmingham, England, 1911	Reading, Pa., 1912	Columbus, Ohio, 1913	Worcester, Mass., experimental filters, July, 1911 to August, 1913	
Kind of filtering material	Slag and broken stone	Slag	Broken stone	Broken stone	Broken stone
Size of filtering material (inches)	1¼ to 2[1]	1½ to 4	1 to 3	½ to 1½	¾ to 2½
Depth of bed (feet)	6.0	5.0	5.3	10.0	10.0
Rate of filtration:					
Million gallons per acre per day	0.865	1.55	1.56	1.71	1.71
Gallons per cubic yard per day	89.0	180.0	182.0	106.0	106.0
(Parts per 1,000,000)					
Suspended matter:					
Filter influent	84.0	130.0[2]	121.0	145.0	145.0
Filter effluent	114.0	54.0[2]	63.0	....	....
Secondary tank effluent	22.0	26.0[2]	65.0	45.0	52.0
Per cent. reduced (total)	73.9	80.0	46.3	69.0	64.1
Oxygen consumed from permanganate:					
Filter influent	110.1[5]	31.0[4]	104.0[3]	115.0[6]	115.0[6]
Filter effluent	37.6	18.0	26.0	....	....
Secondary tank effluent	21.1	13.0	22.0	30.0	32.0
Per cent. reduced (total)	80.8	58.1	78.8	73.9	72.1
Albuminoid ammonia:					
Filter influent	6.1	....	....	6.4	6.4
Filter effluent	....	....	....	....	....
Secondary tank effluent	1.9	....	....	2.3	2.5
Per cent. reduced (total)	68.9	....	....	64.1	61.0
Nitrogen as nitrites and nitrates:					
Filter influent	....	....	....	0.92	0.92
Filter effluent	....	5.3[7]	....	....	....
Secondary tank effluent	18.3	5.6	4.1	6.04	5.17
Relative stability (per cent.):					
Secondary tank effluent	....	98.0	67.0	....	....
Percentage of samples non-putrescible, Secondary tank effluent	....	....	....	89.0	77.5
Bacteria, millions per cc.:					
Filter influent	....	1.30	....	6.48	6.48
Filter effluent	....	0.38	....	....	....
Secondary tank effluent	....	0.54	....	1.06	1.50
Per cent. reduced (total)	....	58.4	....	83.7	77.1

[1] Some fine material on top of some of the beds.
[2] Suspended matter determined and recorded as "turbidity."
[3] Dissolved oxygen consumed during digestion for 24 hours at 37°C.
[4] Digestion for 5 minutes at 100°C.
[5] Digestion for 4 hours at room temperature.
[6] Digestion for 2 minutes at 100°C.
[7] Does not include nitrites.

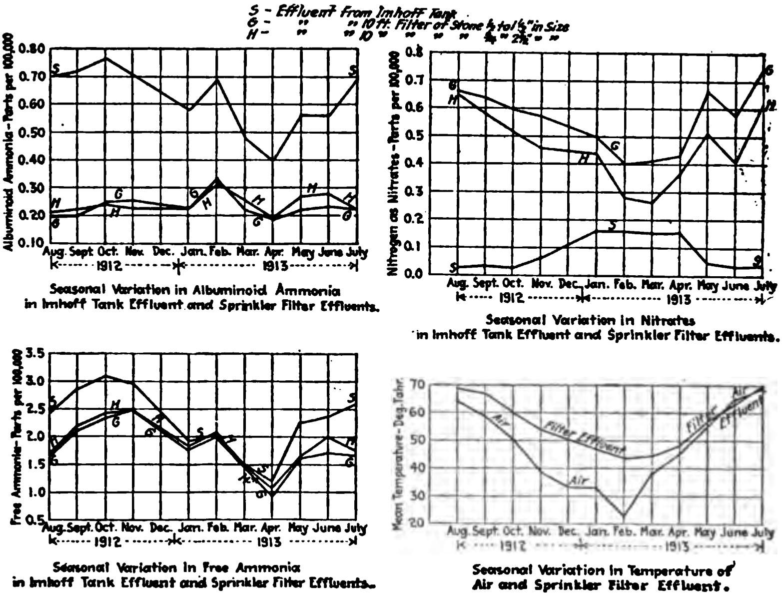


FIG. 173.—Seasonal variations in the purification effected by experimental trickling filters at Worcester, Mass.

FIG. 174.—Ice formation on trickling filters at −10°F., at Worcester, Mass.

cent. of the area was open. The ice formation at this time is shown in Fig. 174.

At Gloversville, N. Y., where the winters are much more severe than at Worcester, it was deemed advisable to house the trickling filter. This was not done until after the winter of 1912–13; although that winter was unusually mild, the ice formation at times covered two-thirds of the area. The filter is now protected during the winter with a temporary roof and sides of rough hemlock lumber. The enclosure cannot be made perfectly tight on account of excessive vapor and bad air.

The Gloversville plant provides for the sedimentation of the trickling filter effluent and subsequent filtration through sand beds. Table 139 gives the average monthly results for 1914 of methylene-blue putrescibility tests of the effluents from the trickling filters, secondary tanks, and sand filters, the samples being incubated at 68°F.

Table 139.—Average Relative Stability of Effluents from Sewage Disposal Plant, Gloversville, N. Y.

Month	Temperature of air, °F.		Percentage of stable samples of effluent from		
	Minimum	Mean	Trickling filters	Secondary settling tanks	Sand filters
January	−25	18.6	35.2	36.5	100
February	−25	11.8	68.0[1]	68.0[1]	100
March	0	27.8	75.0	75.0	....
April	19	48.0	65.0	68.0	100
May	....	....	52.0	58.0	100
June	....	....	55.0	62.0	100
July	....	....	51.0	58.0	100
August	....	....	62.0	64.0	100
September	....	....	57.0	62.0	100
October	....	....	62.0	69.0	100
November	12	38.0	65.0	67.0	100
December	−30	22.0	46.0	51.0	100

[1] Samples were not decolorized at the end of 5 days when they were thrown away.

Capacity of Trickling Filters.—The capacity of trickling filters is dependent upon the strength and character of the applied sewage as well as upon the size and depth of filtering medium. The Royal Commission on Sewage Disposal in its Fifth Report estimated the capacity of coarse filters at approximately 100 to 200 U.S. gal. per day per cubic yard of filtering material, which is equivalent to nearly 1,000,000 to 2,000,000 gal. per acre per day on a bed 6 ft. deep. The

maximum limit set by the Royal Commission might be considered a safe estimate for ordinary domestic sewage in the United States, but for industrial wastes or sewage containing unusual amounts of such wastes much lower rates may be necessary.

Fuller has stated that a fair average loading for a filter 7 ft. deep is 19,000 population per acre. (*Proc. Am. Soc. Mun. Imp.*, 1914.) Trickling filters in the United States have been designed generally for between 2000 and 4000 persons per acre per foot in depth. The authors believe that the former is a safe estimate for treating settled domestic sewage by trickling filters 5 to 10 ft. deep composed of broken stone between 1 and 2 in. in size.

Relative Merits of Trickling Filters and Contact Beds.—The conclusion on this subject reached by the Royal Commission on Sewage Disposal has been quoted on page 27.

At Worcester, Mass., where large quantities of sulphate of iron are present in the sewage, it was concluded that four times as much settled sewage could be treated with satisfactory results by trickling filters as by contact beds and that at least 3 contacts would be required to produce as high nitrification by contact beds as by trickling filters.

With filters of coarse material not subject to disintegration, the evidence seems to indicate that they will be self-cleansing if properly

TABLE 140.—COST OF CONSTRUCTION OF CERTAIN TRICKLING FILTERS BUILT OR PROJECTED IN THE UNITED STATES

Place	Area, acres	Depth of stone, feet	Cost per acre	Cost per cu. yd. of filter	Remarks
Reading, Pa. (*a*)	1.0	6.3–7.0	$37,500	$3.50	Cost of second unit constructed, including dosing equipment and secondary sedimentation tank.
Columbus, O. (*a*)	10.0	5.3	24,040	2.81	Exclusive of conduits and engineering.
Washington, Pa. (*b*)	1.38	6.8	31,700	2.89	Exclusive of engineering.
Gloversville, N. Y. (*b*)	3.07	4.5	32,632	4.50	Exclusive of conduits, roof and engineering.
Fitchburg, Mass. (*b*)	2.0	10.0	59,650	3.70	Exclusive of engineering.
Chicago, Ill. (*c*)		7.0	45,000	3.98+	Including office and laboratory, excluding engineering.
Paterson, N. J. (*c*)		10.0	50,750	3.14	Complete.
Baltimore, Md. (*c*)	12.0	9.0	50,700	3.49	Exclusive of engineering.
East Orange, N. J. (*c*)		7.5	45,000	3.72	Including foundations and dosing apparatus but excluding engineering. Allowance for foundations, $7000.

(*a*) Actual cost.
(*b*) Estimated cost based on quantities from design and contract prices.
(*c*) Estimated cost.

operated, whereas contact beds usually clog periodically. Hence the co..t of treatment by trickling filters is usually much less than that by contact beds.

The effluent from the trickling filter is ordinarily more highly nitrified than the effluent from contact beds and after secondary sedimentation is more uniform in quality than contact bed effluent.

The trickling filter is better adapted for variations in rates of flow than is the contact bed.

The chief advantages in the use of contact beds rather than trickling filters are the relatively low head required, the somewhat simpler method of dosing, minimizing foul odors and avoiding a fly nuisance.

Table 141.—Itemized Cost of Sewage Disposal Plant, Gloversville, N. Y.

	Total cost	Unit cost
Screen chamber substructure	$445.27	
Screen chamber house (total exterior volume, 1350 cu. ft.).	535.00	$0.40 per cubic foot.
Primary settling tanks (total capacity both tanks, 537,000 gal.).	15,761.45	26.10 per 1,000 gal.
Dosing tank (total capacity, 8800 gal.).	1,403.52	160.00 per 1,000 gal.
Trickling filters (3.07 acres, area of stone).	106,560.48	34,700 per acre
Secondary settling tanks (total capacity both tanks, 242,000 gal.).	9,292.31	38.40 per 1,000 gal.
Sludge beds (2.63 acres effective sand area).	9,097.23	2,450 per acre
Sand filter beds (2.72 acres effective sand area).	24,716.77	9,090 per acre
Sludge pump well (total capacity, 16,230 gal.).	1,232.61	75.90 per 1,000 gal.
Sludge pump house (total exterior volume, 1880 cu. ft.).	535.00	0.39 per cubic foot
Sludge pumping machinery	460.00	
Conduits and pipe lines (sewage, effluent, sludge, water).	11,758.85	
Grading, drives, walks, trees, cleaning up, etc.	3,800.78	
Creek deepening and straightening	1,396.30	
Miscellaneous	57.99	
Extras, claims, incidentals, delays and damages.	1,700.00	
Total cost	$188,753.56	

On basis of 3,000,000 gal. of sewage treated daily, $62,900 per 1,000,000 gal.

Cost of Construction.—The actual or estimated costs of various trickling filters built or projected in the United States are shown in Table 140. They vary from $24,000 to $60,000 per acre and from $2.81 to $4.50 per effective cubic yard of filter. Fuller gives the average cost of a trickling filter 7 ft. deep as $45,000 per acre, or $2.37 per capita, based on a population of 19,000 served per acre. (*Proc. Am. Soc. Mun. Imp.*, 1914.) The actual cost of the 10-ft. Fitchburg trickling filter was $58,847 per acre exclusive of excavation, or $2.94 per capita, based on a population of 20,000 per acre of filter. (Hartwell, *Jour. Bos. Soc. C. E.*, vol. ii, 1915, page 221.)

The relation which the cost of trickling filters bears to the costs of the other parts of a trickling filter plant will vary according to the design, as indicated in Tables 141 and 142, showing the actual itemized costs of construction at Gloversville, N. Y., and Fitchburg, Mass. The roof system at Gloversville increased the cost of the trickling filter by approximately $13,336, or $4445 per acre. Of this total, $4406 was for columns, $4550 for beams and $4380 for lumber, etc.

TABLE 142.—ITEMIZED COST OF DIFFERENT FEATURES OF SEWAGE DISPOSAL PLANT, FITCHBURG, MASS.

(Sewage Disposal Commission, Ninth Semi-Annual Report, 1914, page 7)

	Total cost (approximate)	Cost per capita (approximate)
Venturi meter and chamber	$2,942.25	$0.053
Imhoff tanks	56,122.53	1.02
Sludge beds	3,054.81	0.055
Dosing tank and apparatus	10,661.52	0.19
Trickling filters	136,545.53	3.41
Pipe lines	9,668.27	0.17
Overflow chamber	901.93	0.016
Secondary tanks	8,969.62	0.16
Pump house and pump	2,007.64	0.036
Effluent channel	1,328.91	0.024
Roadways	10,710.57	0.19
River improvement	5,122.67	0.093
Bonus paid contractor	5,000.00	0.091
Additional work to be done	10,000.00 est.	0.18
Engineering and inspecting	30,000.00 est.	0.54
Total (approximate)	$293,036.25	6.22

Cost of Operation.—There appear to be few data of the cost of operation of the trickling filters in the United States. In most cases where costs are kept, no attempt has been made to divide the charges among the different parts of the plant.

At Columbus, Ohio, the operation of the entire treatment plant, exclusive of pumping station, for 1913 cost $8286.60; or approximately $2.40 per 1,000,000 gal. of sewage treated during 222 days of the year. (Rept. Div. of Sewage Disposal, 1913.) C. B. Hoover, Chemist in charge, informed the authors that the proportionate cost of operating the different parts of the plant was, approximately, preliminary tanks 4 per cent.; trickling filters, 6 per cent.; final sedimentation tanks, 90 per cent. The comparatively high cost of operating the secondary tanks was probably due to the difficulty of sludge disposal. The sludge from the preliminary tanks was pumped into the river during high stream flow. Hoover has furnished the subdivision of the average cost of operation of the Columbus plant per 1,000,000 gal. of sewage treated, given in Table 143. The "actual cost" is the total annual expenditure for each of the items divided by the millions of gallons treated, while the "cost for time in service" is the expenditure for each of the items during the 222 days of operation divided by the millions of gallons treated.

TABLE 143.—COST OF OPERATION OF SEWAGE TREATMENT PLANT AT COLUMBUS, OHIO, PER 1,000,000 GAL. TREATED

	Supervision	Chemical control	Operation of treatment devices	Care of grounds, etc.	All other items[1]	Total
Actual cost	$0.45	$0.32	$0.92	$0.47	$0.43	$2.59
Cost for time in service	0.29	0.21	0.42	0.32	0.30	1.54

[1] Includes transportation, heat, repairs, printing, supplies, light and telephone service.

At Reading, Pa., the net expenditure for maintenance and operation of the sewage pumping and disposal works for 1912 was $15,470.24, equivalent to $9.13 per 1,000,000 gal. of sewage treated. (Rept. City Engineer, 1912.) City Engineer Ulrich advised the authors that the cost of operation of the disposal plant alone for that year was $5215.10, which is equivalent to $3.08 per 1,000,000 gal. treated. E. Sherman Chase, formerly chemist in charge, stated that the labor in connection with the trickling filters was performed by three men working in 8-hour shifts, who act as watchmen, collect samples for analysis and care for the laboratory and grounds. These men are paid $2 per day, so that the labor cost is a little over $1 per 1,000,000 gal. sewage filtered. (*Engineering News*, August 22, 1912.)

Calvin W. Hendrick, Chief Engineer of the Sewerage Commission of Baltimore, stated that the cost of operation of the Baltimore sewage treatment plant, with 12 acres of trickling filters, when working up to its capacity, will probably be between $1.50 and $2 per 1,000,000

gal. The organization at this plant Mr. Hendrick gave as follows: 1 division engineer, who also supervises construction work; 1 mechanical engineer; 1 chemist and bacteriologist; 1 assistant chemist; 3 operating engineers; 1 relief engineer and 4 oilers for the power plant; 1 machinist; 1 carpenter; 1 foreman for laborers; and 12 to 20 laborers.

The organization at the Pennypack Creek disposal works, Philadelphia, Pa., designed to treat 2,000,000 gal. daily, was stated by George S. Webster, Chief Engineer of the Bureau of Surveys, as follows: The assistant engineer of the Sewage Disposal Division has supervision of the operation of the plant, which requires only a small part of his time, and an assistant has immediate charge of maintenance, supplies and records. The force at the plant consists of an operator on duty every day, having immediate charge of the operation, sampling, etc., 4 assistant operators working 8 hours a day, 6 days a week, a watchman for night duty, and a laborer for day duty, such as handling sludge, caring for lawns, shrubbery, etc. The analytical work is done partly at the Bureau of Water laboratory nearby and partly at the Bureau of Surveys Laboratory at the City Hall.

The costs of operating different parts of the plant at Gloversville, N. Y., have been very carefully kept under the direction of H. J. Hanmer, City Engineer. The itemized cost for 1913 and 1914 is given in Table 144. The cost of operating the trickling filters alone constitutes roughly 15 to 25 per cent. of the total for the entire plant. The cost of removing and replacing the roof and sides of the building in which the filter is housed during winter constitutes a substantial part of the trickling filter maintenance charges. The remainder is occasioned by nozzle clogging. About 60 nozzles, or approximately 10 per cent. of those in use, are cleaned each day.

The cost of operation of trickling filter plants, per 1,000,000 gal. of sewage treated, other conditions being equal, will decrease with increasing size of plant. Estimates made by the authors in connection with the joint disposal of sewage from several municipalities in New Jersey ranged from $5.19 per 1,000,000 gal. for an estimated flow of 4,400,000 gal. daily to $2.92 for 14,300,000 gal. E. J. Fort, Chief Engineer of Sewers of Brooklyn, estimated the cost, including interest and depreciation or sinking fund, at $13.81 per 1,000,000 gal. with a flow of 5,000,000 gal. daily, $11.41 for a rate of 10,000,000 gal., $9.76 for 20,000,000 gal. and $9.50 for 30,000,000 gal.

Thomas Pealer, Borough Engineer of Indiana, Pa., furnished the following information concerning the sewage disposal plant at Indiana, Pa., comprising screen chamber, septic tanks and a trickling filter 220 × 100 × 5½ ft. deep, of ½ to 3½-in. broken stone, with dosing tank and fixed nozzles. It serves 8000 persons and treats 500,000 to 1,000,000

gal. daily of domestic sewage from separate sewers. This plant cost $40,000 and is operated by 1 man at a cost of $750 per annum.

TABLE 144.—COST OF OPERATION OF SEWAGE TREATMENT WORKS, GLOVERSVILLE, N. Y.

	1913	1914
Supervision by city engineer	....	$600.00
Operation of screens, etc	$223.37	574.00
Sludge pumping:		
Labor and repairs	226.00	297.00
Electric power	318.34	214.05
Maintenance of trickling filters:		
Nozzles	897.28	395.00
Removing and replacing covering	278.25	359.00
Maintenance of sludge beds	1,312.46	1,235.00
Cleaning troughs of secondary tanks	35.50	19.63
Maintenance of sand filters	440.33	315.00
Maintenance of grounds	399.97	163.00
Miscellaneous work	169.87	365.00
Chloride of lime and other supplies	835.96	664.27
Telephone	....	42.00
Cleaning and repairing east primary settling tank (unusual item)	....	709.00
Total cost for year	$5,137.33	$5,951.95
Cost per 1,000,000 gal. treated, average flow 2,750,000 gal. daily	5.16	5.92
Cost per capita based on estimated population.	0.24	0.27
Estimated population	21,600	21,800

The plant at Chambersburg, Pa., as described by Frank H. Clutz, Borough Engineer, consists of Imhoff tanks, a trickling filter 160 × 125 × 7 ft. deep, of 1½ to 3½-in. limestone, with dosing tank and fixed nozzles, and secondary sedimentation tanks. The entire plant cost $46,595.25, exclusive of land, the cost of the trickling filter alone being estimated at $18,500. This plant cares for the sewage from about 5400 persons of the town population of about 13,500, and the average flow of sewage treated, including ground water, is about 1,400,000 gal. per day. Two men are regularly employed, one during the day and one at night, and occasional assistance is required. The cost of operation, maintenance and improvements for 1914 was $4302.54.

The sewage of the State Hospital for the Insane, Norristown, Pa., Oscar L. Schwartz, Steward, is treated by a coarse screen, sedimentation tank, trickling filter 100 × 173 × 6½ ft. deep, of 1½ to 3½-in. lime-

stone, with dosing tank and fixed nozzles, and final sedimentation tanks. The number of persons at the hospital is 3500 and the quantity of sewage treated is 575,000 gal. per day. Two men are employed at this plant and the annual cost of operation is estimated at $1290.

The sewage disposal plant of the State Hospital for the Insane at Warren, Pa., according to Albright & Mebus, consists of an Imhoff tank, a trickling filter 95 × 99½ × 7½ ft. average depth, of stone 2 to 3½ in. in size, with dosing tank and fixed nozzles, and a final sedimentation tank. It serves about 1800 persons and treats about 270,000 gal. per day. The cost of construction was $12,800 or $59,000 per acre. One man is employed about 6 hours each day in caring for this plant.

The trickling filter plant at the United States Naval Training Station, Great Lakes, Ill., according to Lieut. J. B. Earle, Public Works Officer, consists of preliminary septic tanks and roughing filters and 2 trickling filters, each 20 × 60 × 7 ft. 4-in. deep, of ¼ to ¾-in. stone, dosed by splash-plate distributors. The plant serves 900 people and treats 300,000 gal. of sewage per day. The cost of construction of the filters was $35,939.50 and the annual cost of operation is estimated at $300.

Dr. L. Rosenburg, Superintendent of the Montefiore Home County Sanitarium, Bedford Hills, N. Y., reports that the sewage disposal plant at this institution, accommodating 245 persons, consists of septic tanks, 3 small trickling filters of 2-in. stone with 1 nozzle each, and a settling tank for the effluent. This plant cost $10,000 and the cost of operation is stated to be negligible, although the engineer visits the plant each day.

CHAPTER XVI

INTERMITTENT SAND FILTRATION

Where a deposit of free sand or sand and gravel is available in place, it may be used for intermittent filtration by simply grading the surface to receive the sewage. Loam, subsoil and silt are not desirable as filtering media on account of their tendency to hold water by capillarity, preventing successful aeration of the bed, except when very low rates of filtration are used, such as those employed in broad irrigation. Clay and cementitious sands or other comparatively impervious materials are useless for filters.

The removal of loam and subsoil is necessary if any considerable quantity of sewage is to be purified upon beds of a given area. Relative expense will probably determine the extent to which it is desirable to remove the subsoil. Where there are trees, organic matter will be found around their roots at a considerably greater depth than where there are no trees, and care must be exercised to remove this in grubbing out tree roots. Similarly, in gravelly soils containing many large stones, fine sandy material may be found surrounding the stones. Therefore, beds built in such material are not likely to be so homogeneous as those built in ground made up of more uniform material.

The limit for excavation may be determined in several ways: first, by color; second, by loss of weight on ignition, due to the volatilization of the organic matter; third, by taking a small portion of the sand in a glass of water, shaking thoroughly, and permitting it to subside, the amount of organic matter and fine sand found upon the top of the sand, when the material has settled, furnishing a ready guide as to the relative content of objectionable matter.

Uniformity of Material Desirable.—Stratification, or the presence in an otherwise uniform and satisfactory material of sand of different sizes or of cementitious character, is objectionable. When sewage is run onto a bed of uniform material, the suspended matter is arrested upon or near the surface, the water gradually passing through the bed at a comparatively uniform rate without any tendency to clog except at the surface. If the material is stratified, with the coarser sand on top, the bed is likely to become clogged by a film of organic matter on the surface of the fine sand below. This may be caused in part by the passage of very fine suspended matter through the coarser sand and its retention upon the surface of the fine stratum, and also probably

by the formation of an organic growth there, due to difference between the quantities of oxygen and water contained in the coarse and fine sands. If the finer material is on top, while there will be no tendency for the fine suspended matter to form a clogging film on the surface of the coarser sand, there may be an accumulation of oxide of iron there due to the difference in the quantities of oxygen present in the two strata. A precipitation of oxide of iron may take place throughout the stratum of coarse material, and if this sand is underlaid with a stratum of fine sand, a film of oxide of iron will form upon the surface of the finer material. An interstratified layer of fine material may act as an air seal, due to capillary action, and thus prevent the satisfactory aeration of the lower portion of the bed.

Effective Size and Uniformity Coefficient of Sand.—As a ready means of determining the comparative value of sands, the standard developed by Allen Hazen when in the employ of the Massachusetts State Board of Health (1892 Report, page 541), defining the "effective size" and the "uniformity coefficient," has been found most useful.

"As a result of experiments made at the Lawrence Experiment Station we have a standard by which we can definitely compare various sands. The size of a sand grain is uniformly taken as the diameter of a sphere of equal volume, regardless of its shape. As a result of numerous measurements of grains of Lawrence sands, it is found that when the diameter, as given above, is 1, the 3 axes of the grain, selecting the longest possible and taking the other two at right angles to it, are, on an average, 1.38, 1.05, and 0.69, respectively, and the mean diameter is equal to the cube root of their product.[1]

"It was also found that in mixed materials containing particles of various sizes, the water is forced to go around the larger particles and through the finer portions which occupy the intervening spaces, so that it is the finest portion which mainly determines the character of the sand for filtration. As a provisional basis which best accounts for the known facts, the size of grain such that 10 per cent. by weight of the particles are smaller and 90 per cent. larger than itself is considered to be the effective size. The size so calculated is uniformly referred to in speaking of the size of grain in this work.

"Another important point in regard to a material is its degree of uniformity, whether the particles are mainly of the same size or whether there is a great range in their diameters. This is shown by the uniformity coefficient, a term used to designate the ratio of the size of grain which has 60

[1] Fuller made a study of about 200 grains of sand from the Dayton bar in the Ohio River near Cincinnati. The long, middle and short axes of grains retained on a No. 20 sieve (with 0.96-mm. openings) were measured with a micrometer caliper and found to average 2.32, 1.67 and 1.00 in relative length respectively. The long, middle and short axes of the grains passing a No. 20 sieve were found to average 1.98, 1.53 and 1.00 respectively. The error in assuming the middle diameter of the grain as equal to the diameter of a sphere of equal volume is 6 and 5 per cent. in the two cases. (Report on Water Purification, Cincinnati, 1899.)

per cent. of the sample finer than itself to the size which has 10 per cent. finer than itself."

Sieving.—Sieves are usually made to nest one over the other, the coarsest on top, and the nest is designed to fit in a mechanical shaker. The screens in the bottoms of sieves are rated with thoroughly dried sand. In doing this, about ¼ lb. of this sand is placed in the top sieve and the nest is shaken until only a little sand remains to pass through each screen. The sieves are separated and each is shaken by hand over a sheet of paper, the grains that fall through being preserved carefully, as they give the measure of the screen through which they pass. Their average weight is obtained by weighing a number on a laboratory balance and then computing the diameter by the formula:

$$\text{Diam. in mm.} = \sqrt[3]{\frac{6}{\text{Sp. gr.} \times \pi}}\sqrt[3]{w} = 0.9\sqrt[3]{\text{weight in milligrams}}$$

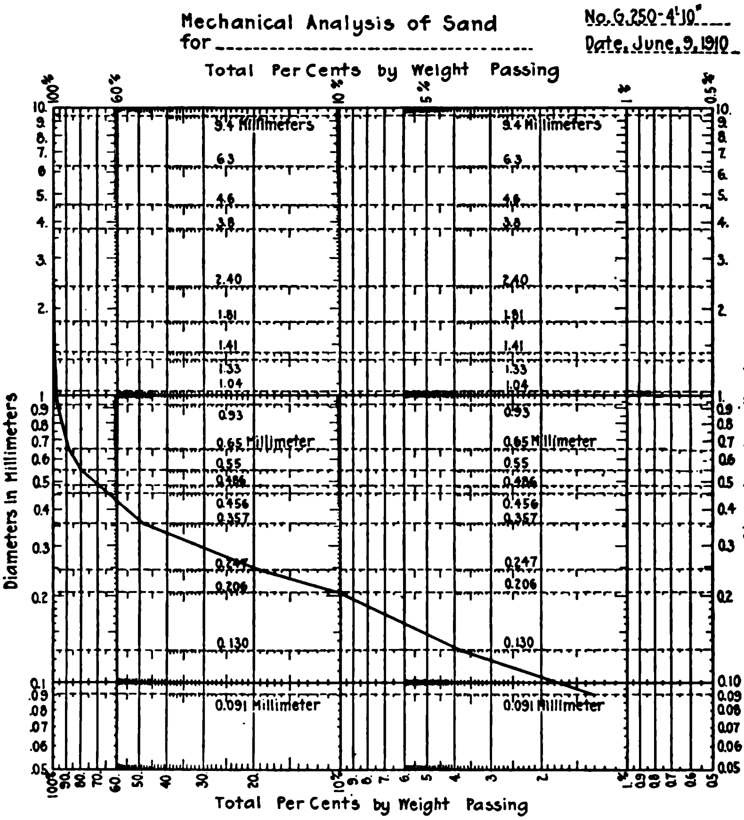
In this expression w is the weight of the grains in milligrams.

The openings in screens of the same nominal number of meshes per linear inch vary with different makes of wire cloth and even with cloth woven by the same maker at different times. In a general way, a 200-mesh sieve will give a 0.10-mm. separation; 140-mesh, 0.13 mm.; 100-mesh, 0.17 mm., 50-mesh, 0.33-mm.; 40-mesh, 0.48 mm.; 30-mesh, 0.63 mm.; and 20-mesh, 0.95 mm.

About ½ lb. of sand is used for a mechanical analysis or more where the sand is very uneven in size. Where the samples contain dust which does not become detached from the sand grains freely, or where the dust adheres to the wires of the screens, it is sometimes desirable to place the full sample on the finest screen first and shake it, then place the sample on the next coarser screen, and shake that screen independently, then transfer the sand to the next coarser screen and shake again independently. After the 80-mesh screen has been used in this way, the sample should be placed in the coarsest sieve, the sieves should be nested, and the test made by shaking the nest until the sand has been given, in the separate sieves and the nest, the total number of shakes necessary to clean and separate it. It is well to adopt a definite number of shakes in a given time, as 200 in 75 seconds. The number and speed required will vary with different machines and different quantities and kinds of sand. An examination will indicate whether the sand has been properly segregated into its component sizes.

Elutriation.—For the determination of the finest material, methods of elutriation must be resorted to. Such materials are, however, practically valueless for the filtration of sewage. It is not practicable to extend sieve operations to particles much smaller than 0.10 mm. (passing a sieve with about 200 meshes to 1 in.). The portion finer than 0.10

mm. contains most of the organic matter; this should be removed by ignition, a low heat being required to prevent decomposition of mineral



Mechanical Analysis

Date of Collection, June 9, 1910

Date of Analysis, June 15, 1910

Size of Size: Sieve	Size of Size: M.M.	Amt. Passing: Grams	Amt. Passing: Per Cent
200.	0.091	2.6	1.3
140.	0.130	7.5	3.8
100	0.206	20.0	10.3
80.	0.247	38.3	19.6
60.	0.357	94.0	48.2
Cu. 40.	0.456		
40.	0.486	136.5	70.0
35.	0.55	157.1	80.6
30	0.65	174.5	89.5
Cu. 26	0.66		
25	0.93	188.4	96.7
20.	1.04	191.5	98.3
16.	1.41	193.7	99.2
0.057"	1.33		
0.079"	1.81		
0.103"	2.40	194.3	99.7
0.156"	3.8		
0.187"	4.6		
0.25"	6.3		
0.375"	9.4		
Total		194.3	99.7
Loss		0.7	0.3
Amount Sieved		195.0	100.0

Woven Brass Wire (sieves 200. to 16.); Punched Metal (0.057" to 0.375")

60% Finer than	0.42 mm.
10% Finer - Eff. Size	0.20 mm.
Uniformity Coefficient	2.1

Analyst . . S.A.W. . . . . . .

FIG. 175.—Form for recording mechanical analyses of sand.

matter. Loss in weight is calculated as organic matter and considered as below 0.01 mm. diameter.

Five grams of the ignited material are placed in a beaker of distilled

water at 20°C. and mixed by blowing air into it. The mixed sand and water are allowed to settle for 15 seconds and then rapidly decanted. Temperature has a marked effect on the rapidity of settling. This operation is repeated three times. The weight remaining is considered as above 0.08 mm. diameter. This process is repeated with the finer material previously decanted, except that settlement is allowed for 1 minute. The weight remaining after this second operation is considered as above 0.04 mm. diameter. (Rept. Mass. Bd. Health, 1892, page 543.)

The accompanying diagram, Fig. 175, shows the form used by the authors for recording the results of mechanical analyses of sand. The effective size and uniformity coefficient are ascertained from the curve of the percentages by weight for the different diameters of sand grains. Coarse material tends to increase the "effective size" so that a sample containing an objectionable amount of fine material may appear to have a reasonably effective size. It is often important to take such facts into consideration when judging of the character of sand with high uniformity coefficients.

Desirable Limits of Effective Size.—It is desirable for the effective size of sand used in intermittent filter beds to lie between 0.20 and 0.35 mm., although good work may be done by materials outside these limits. Materials of 0.10 to 0.20 mm. will give admirable effluents, but cannot pass such large quantities of sewage and tend to clog somewhat more quickly. The coarser materials may lead to difficulties in satisfactory distribution of the sewage over the bed and less efficient purification. Sands with effective sizes even lower than 0.10 mm. are being successfully used in Massachusetts, and some good filters contain sand of an effective size as low as 0.03 mm. The deleterious effect of even a small proportion of very fine sand in filter beds has led some engineers to the opinion that it may be economical in certain cases to go so far as to remove the finest material by washing. In fact it has even been specified in some cases that not over 1 per cent. of the sand grains shall be less than 0.13 mm. or thereabouts in diameter.

The nearer the uniformity coefficient is to unity the more desirable will be the sand. A high uniformity coefficient means that there is a considerable amount of inert material in the bed. Nevertheless good work has been done by such beds. The uniformity coefficient of the majority of beds graded *in situ* in New England is between 3 and 15.

Depth of Bed.—The greatest bacterial activity of the filter is in the upper portion of the bed, particularly in the top 6 to 12 in. Less purification is effected by each successive foot in depth. Nevertheless, it has been found advantageous practically to build filter beds 3 ft. and preferably 4 ft. deep above the underdrains, in order to prevent the sewage from breaking through the bed and reaching the underdrains in an im-

properly purified state. The greater depth has thus a steadying effect upon the purification effected by the bed, and the efficiency of the process.

Clark stated in the 1908 report of the Massachusetts State Board of Health, page 304, that, other things being equal, the filters of greater depth gave effluents of higher purification than those of less depth. With coarse sands, 0.25 mm. effective size or larger, 4 or 5 ft. in depth of bed was desirable. A filter 10 ft. deep gave somewhat better effluent, but not markedly so. Shallow filters (2 ft.) of coarse sand gave fair results when operated at low rates.

Filters composed of fine sand must be deep enough to overcome capillarity which, at 0.04 mm. effective size or larger, will raise water 2 ft. more or less in the bed, so as to provide unsaturated sand layers near the surface. With filters composed of fine sand, when the matter of capillarity is taken care of by providing sufficient depth, the effluent is better than that from a filter of coarse sand of equal depth. These filters of fine sand are, however, operated with more difficulty.

RATE OF FILTRATION

The volume of sewage which can be purified per acre of any given filter bed depends primarily upon the amount and character of the organic matter present in the sewage. If an excessive amount of sewage be applied to the bed the filter will store up the organic matter, the aeration will be inadequate, and bacterial activity will decrease until the bed becomes clogged. Its use will then have to be suspended until the liquid has drained off thoroughly and a period of rest has been allowed, during which the accumulated organic matter becomes oxidized.

The effect of the size of the sand grains on the rate of filtration is shown by the experience with 7 filters operated for many years at the Lawrence Experiment Station. At the close of 1913 these beds had been in service for 19 to 26 years, receiving the sewage delivered to the station without preliminary treatment. The general record of the tests is given in Table 145. Except for filter 10, the beds had 6 in. of gravel and wooden underdrains below the sand. The only gravel in filter 10 was directly above the outlet, and there were no underdrains. A partition extending 3 ft. below the surface was placed in the sand of this filter, separating the quarter of the surface farthest removed from the outlet from the remainder, and to this quarter all the sewage was applied. These beds were operated at various rates for a number of years, but later the attempt was made to apply only so much organic matter as each filter could assimilate, the rates for 1913 given in Table 145 being practically the maximum at which such beds could then be worked. Filter 10 was the only one which did not show satisfactory

results as respects nitrification, but in the latter part of 1913, after the rate was reduced to 40,000 gal. per acre daily on the quarter of the bed dosed, or 10,000 gal. per acre daily on the entire filter area, there was a marked improvement in the quality of the effluent. All of the beds were carefully tended throughout their period of service.

TABLE 145.—LONG-TIME FILTRATION EXPERIMENTS AT LAWRENCE EXPERIMENT STATION

(From Report of Massachusetts State Board of Health, 1913)

Filter number	Depth of sand, ft.	Effective size of sand, mm.	Operation began	Rate of operation, gallons per acre daily[1]		
				Initial	Maximum	1913
4	5	0.04	Dec. 19, 1887	28,700	41,800	20,900[2]
2	5	0.08	Dec. 19, 1887	28,200	50,800	38,500
9a	5	0.17	Nov. 18, 1890	110,000	111,700	47,400
5c	5	0.22	July 20, 1905	54,200	54,200	47,700
6	3.7	0.35	Jan. 12, 1888	39,500	85,500	46,600
10	5	0.35	July 18, 1894	160,000	160,000	12,500[3]
1	5	0.48	Jan. 10, 1888	53,400	124,100	47,900

[1] Operated 6 days in the week.
[2] Three times each week.
[3] Fifty thousand gallons applied daily to one-fourth of the surface.

The effect of preliminary treatment of the sewage on the rate of filtration has been studied at the Lawrence Experiment Station with the help of 4 filters containing 5 ft. of sand of 0.25 mm. effective size. These beds were dosed at rates giving, as closely as possible, the same amount of organic matter to each. The filters were put in operation early in 1911 and the tests were unfinished at the close of 1913. The results available at this time (1915) are given in Table 146. The average rates of dosing given in the table are the averages of wide fluctuations, for considerable difficulty was experienced in keeping the amount of organic matter applied to all filters the same; in fact, a monthly plus-and-minus nitrogen account is kept for each bed and fluctuations below or above the standard are smoothed out by increasing or decreasing the rate of application. The 1913 results of operation were discussed by Clark and Gage in the report of the State Board for that year as follows:

"All of these effluents were highly nitrified throughout the year, the amount of nitrates produced being greater than the alkalinity of the sewage could care for, with the result that the effluents were acid during the greater part of the year. As was the case during 1912, the effluent from the filter receiving untreated sewage contained considerably more nitrates than that from the filters receiving clarified sewage, while the effluent from the filter receiving chemically precipitated sewage was more highly nitrified than that from the filters receiving strained or settled sewage. On the

basis of the total amount of nitrogen oxidized to nitrates, however, the filters receiving the precipitated and the settled sewage have been more active during the past 2 years" (page 315).

TABLE 146.—RESULTS OF APPLYING DIFFERENT SEWAGES TO SIMILAR FILTERS AT DIFFERENT RATES SO AS TO FURNISH THE SAME AMOUNT OF ORGANIC MATTER TO EACH FILTER

(Parts per 1,000,000. From Report of Massachusetts State Board of Health, 1913)

Filter number	429		430		431		432	
	Untreated sewage applied	Effluent	Settled sewage applied	Effluent	Strained sewage applied	Effluent	Chem. prec. sewage applied	Effluent
Ammonia free	48.5	3.006	38.1	2.358	38.6	2.943	49.3	7.666
Ammonia, albuminoid, total.	7.1	0.262	4.4	0.339	4.0	0.320	4.1	0.417
Ammonia, albuminoid, in solution.	4.0	......	3.2	......	3.0	......	2.8	......
Kjeldahl nitrogen, total.	13.4	......	7.9	......	7.5	......	7.6	......
Kjeldahl nitrogen, in sol.	7.6	......	6.0	......	5.7	......	5.4	......
Oxygen consumed	46.8	2.8	30.7	3.3	27.8	3.4	26.2	3.6
Nitrogen as nitrates	......	49.9	......	36.3	......	39.2	......	42.4
Nitrogen as nitrites	......	0.008	......	0.005	......	0.008	......	0.030
Solids, unfiltered, total	714	......	566	......	548	......	525	......
Solids, unfiltered, loss on ignition.	316	......	207	......	191	......	169	......
Solids, unfiltered, fixed	398	......	359	......	357	......	356	......
Solids, filtered, total	480	......	490	......	492	......	461	......
Solids, filtered, loss on ign.	154	......	156	......	149	......	122	......
Solids, filtered, fixed	326	......	334	......	343	......	339	......
Solids, in suspension, total	234	......	76	......	56	......	64	......
Solids, in suspension, loss on ignition.	162	......	51	......	42	......	47	......
Solids, in suspension, fixed.	72	......	25	......	14	......	17	......
Gallons per acre daily,								
1911 average	84,700	......	122,200	......	127,200	......	155,900	......
1912 average	79,500	......	110,200	......	124,200	......	126,600	......
1913 average	79,800	......	145,800	......	120,300	......	140,900	......

The experiments made at the Columbus, Ohio, sewage experiment station in 1905, developed the fact that screened Columbus sewage, from which one-fourth of the suspended matter had been removed, could be filtered on 3-ft. beds of sand of an effective size of 0.25 mm. at a rate of 100,000 gal. per acre per day, with a removal of 90 per cent. of organic matter and 98 per cent. of bacteria. (Report on Sewage Purifica-

tion, Johnson, page 166.) Septic tank effluents applied to the sand filters did not produce as successful results as did those from plain sedimentation tanks. This was supposed to be due principally to the fact that the evoluton of gas in the septic tanks caused fine particles of sludge to be carried over on to the sand filters, thus clogging them.

Conditions of Experiments and Practice Differ.—In practically all experiments the volume of sewage applied to a given area of filter has been determined by and varied with the ability of the bed to satisfactorily treat it. While there have been fluctuations in the quantity applied, it has usually been uniform for prolonged periods. In practice, on the other hand, the flow of sewage varies greatly, as discussed at length in Chapter V, Volume I. For example, at Marlboro, Mass. (population in 1900, 13,609, separate sewers) the average daily flow in September was 587,000 gal. whereas the maximum was 2,028,000 gal. in March, and the maximum flow for a single day, was 3,300,000 in 1903. (Report Mass. St. Bd. Health, 1903, page 388. The average for the year was 1,100,000 gal. per day.)

A practical operating plant, designed to handle the average flow, must be able to care for the maximum rate also; otherwise the surplus must be by-passed. Thus, at Marlboro, the load for March was about double the average for the year and nearly four times that for September. Furthermore, the high flows at Massachusetts plants come at a time when the beds are, generally speaking, in poor condition, that is, at the end of the winter before the spring cleaning is done and when, if ever, aeration is likely to be defective and the surface poorly prepared to admit large quantities of water to the beds.

The surface of an experimental filter is likely to be much more efficiently cared for than that of large plants. With a hand rake the operator can, in a few minutes, thoroughly clean or loosen the entire surface of his experimental bed, but on a large plant such work consumes a number of days or even weeks. The effect of a storm in compacting the sand of an experimental filter can be remedied in a few minutes, while a sudden shower may cause damage to a large filter plant which will require a week or longer to repair. Experimental plants are either protected during the winter or the effect of ice can be easily overcome, whereas a large plant may have a number of its filters actually put out of service by freezing, not to be returned to use until the beds have thawed out in the spring. All of these and many ot her difficulties encountered in practical operation tend to make the results of experiments misleading, and such results should be properly discounted by those designing plants.

Dosing the Filters.—The rate at which the sewage is applied to the filters in Massachusetts has been fairly well standardized by experience. In 1911, Barbour stated his practice as follows:

"The size of dose may be changed but that usually applied is equivalent to a little more than 1 in. in depth on the sand surface, and experience has proved that a rate of discharge equal to 1 cu. ft. per second for each 5000 sq. ft. of area will effect, on the ordinary sand bed, good distribution" (*Jour. Assoc. Eng. Socs.*, 1911, vol. xlvii, page 61).

The authors' practice is to apply the sewage at the same rate, 1 cu. ft. per second per 5000 sq. ft. of area, but they aim to cover the bed to an average depth of 3 in. This is because the average bed has an uneven surface and 3 in. depth of sewage seems necessary to make every square foot serviceable. A dose covering a 1-acre bed 1 in. deep is equivalent to approximately 27,000 gal. per acre.

The frequency of dosing depends upon the capacity of the bed and the size of the individual dose. If a heavy dose, such as one 3 in. deep, is applied to a bed having a nominal capacity of about 30,000 gal. per acre, the bed should be dosed only once in three days. In practice, however, the frequency is largely governed by the quantity of sewage produced at the time.

SHAPE, SIZE AND GROUPING OF BEDS

The size and shape of the beds will be determined by the amount of sewage to be handled and the topographic conditions. In the larger plants rectangular or square beds having an area from ¾ acre to 1 acre, have generally proved most desirable. In smaller plants the size may be much smaller in order to avoid throwing a large proportion of the area out of use when cleaning beds and to facilitate dosing without storing the sewage too long.

The areas of some of the leading Massachusetts filters are given in Table 147. The area of filters provided varies from 1 acre for 500 persons, to 1 acre for 2500 persons of total population, omitting Leicester. Probably in no case is the entire population served by the sewer system. Some of the plants are doing excellent work while others are apparently overloaded, poorly operated or otherwise prevented from attaining efficiency. Experience indicates that the settled sewage of 750 persons, excluding storm water, is a reasonable load for 1 acre of filters in Massachusetts, although when scientifically operated filters may satisfactorily carry a somewhat greater load, under favorable conditions. As a matter of fact it is possible to treat a very much greater quantity during warm dry weather but the working plant must be so designed as to care for the sewage under the most unfavorable conditions. It should have sufficient capacity to allow 1 or 2 beds at a time to be cut out for necessary cleaning or repairs. At 100 gal. per person per day, 750 persons per acre is a load equivalent to 75,000 gal. per acre daily.

Except in very small plants, intermittent filter beds are rectangular in shape when the topography permits. The cost of embankments, which are usually made of the material stripped from the beds, is so small that it is not often a factor to be considered in the determination of the shape.

The underdrainage system and the distribution of sewage over the bed are usually of most importance in determining the shape. With rectangular beds it is practicable to flood them from the 4 corners satisfactorily, even when they are of large size, but the underdrainage is likely to cost more than when long beds are used of such width that two can be drained by laterals running to a main drain laid in the embankment between beds. The distribution of sewage over long beds is not so uniform as it is over square beds unless troughs are used, Fig. 178, page 643, which operators dislike.

As a general thing, several arrangements of beds, distributing conduits, drains and roads are practicable, and some preliminary studies will be needed to determine which is best. The cost of the whole installation rather than that of 1 or 2 beds should be the deciding factor, since main drains or main carriers may prove unexpectedly expensive if judged by an examination of the needs of 1 or 2 beds.

SURFACE AND BANKS OF BEDS

Surface of Bed.—Filter beds are generally graded substantially level. There is little advantage in sloping the surface under ordinary conditions, for if the discharge of sewage on to the bed be rapid, satisfactory distribution will be obtained with a level bed. The treatment of the surface of the bed for the winter is described hereafter under winter operation of beds.

Banks and Roads.—For convenience of access and for the removal of surface deposits it is necessary to provide roadways between successive rows of filters. For these roadway embankments a top width of about 8 ft. has been found to be sufficient. The height of the embankment over the bed will be determined by the fall required for the main sewage distributors, which can be laid with shallow cover, however, on account of the warmth of the sewage carried by them, 24 in. in depth usually being sufficient cover. The side slopes of these embankments should be made steep, as a matter of economy. Ordinarily a slope of 1:1 can be maintained but 1 vertical to 1½ horizontal is preferable. The subsoil and loam stripped from the surface of the beds is used for the embankments, which are grassed over to reduce the cost of maintenance.

A sloping driveway should be provided, leading into each filter bed. It will be found convenient to group these driveways in such a way as to lead from the roadway into 4 beds at their adjacent corners.

TABLE 147.—INTERMITTENT FILTERS OF MASSACHUSETTS TOWNS AND CITIES
(From Report of State Board of Health, 1913, pages 367, 368)

Place	Population, 1910	Sewage, gal. per day	Filter beds: Number[1]	Filter beds: Net area, acres	Rate, gal per acre per day	Persons per acre	Underdrains: Depth, feet	Underdrains: Spacing, feet	Filtering material
Amherst	5,112	375,000	6	2.00	188,000	2,556	4–4.5	25	Rather fine sand found in place.
Andover	7,301	350,000	20	3.65	96,000	2,000	4	20	Fair sand, small quantity of gravel, practically all handled in construction.
Attleboro	16,215	250,000	26	15.50	16,000	1,046	4–7	35	Excellent sand and gravel found in place.
Brockton	56,878	2,175,000[2]	37	37.00	59,000	1,536	5.5	30	Good sand and gravel found in place.
Clinton	13,075	1,090,000[3]	27	26.23	42,000	498	8	60–70	Good sand and gravel found in place.
Concord	6,421	422,000[3]	4	3.30	128,000	1,945	none	....	Good sand underlaid with gravel found in place.
Framingham	12,948	700,000[3]	20	20.75	34,000	623	...[4]..	....	Surface material badly impregnated with organic matter; good underlying sand. Beds prepared by removal of trees and stumps and leveling where necessary.
Gardner									
Gardner } Templeton }	14,699	{ 135,000	20	2.50	54,000	} 1,175	5	20	Good sand, handled in construction.
		{ 600,000	26	10.00	60,000		3–4	20–30	Coarse sand, handled in construction.
Hopedale	2,188	150,000[3]	7	3.25	46,000	674	4	35–60	Some good sand, some rather fine sand, some ledge; material found in place.
Hudson	6,743	276,000	23	9.00	31,000	750	5–6	50–100	Good sand and gravel found in place.
Leicester	3,237	70,000	8	0.36	194,000	9,000	4	8	Hard, compact sand found in place.
Marion	1,460	110,000[3]	8	0.66	167,000	2,210	5	....	Mostly good sand, with pockets of fine sand and some ledge; mostly found in place.
Marlboro	14,579	750,000	33	20.90	36,000	698	4.5–6	30–50	Rather fine sand found in place.
Milford	13,055	513,000	15	9.30	55,000	1,405	5	40	Rather fine sand found in place.
Natick	9,866	720,000[3]	14	12.60	57,000	782	6	36	Good sand with strata of very fine sand in places; found in place.
North Attleboro	9,562	600,000	16	7.00	86,000	1,366	....	....	Very coarse sand and gravel.
Northbridge	8,807	244,000	24	6.00	41,000	1,468	4	50–75	Coarse sand and gravel found in place.
Norwood	8,014	500,000	6	6.64[5]	75,000	1,206	4–6	40	Good sand and gravel found in place.
Pittsfield	32,121	1,903,000[3]	35[6]	25.90	74,000	1,240	4	35	Good sand and gravel found in place.
Southbridge	12,592	900,000	11	8.50	106,000	1,481	4	40	Fair sand and gravel; some found in place, considerable quantity handled.
Spencer	6,740	450,000	12	9.30	48,000	725	...[7]..	....	Good sand and gravel, mostly found in place.
Westboro	5,446	425,000	12	3.80	73,000	1,435	5	30–40	Good sand and gravel handled during construction.
Worcester	145,986	3,860,000[8]	75	74.30	52,000	....	4–6	35–50	Good sand and gravel, largely found in place.

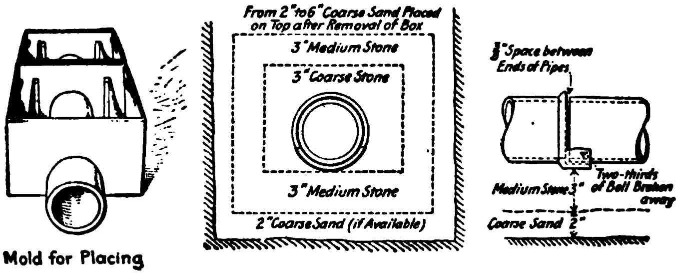
[1] Not including sludge beds. [2] Amount pumped to filters; about 800,000 gal. daily treated during October and November by half-acre trickling filter, sedimentation basin and sand filter. [3] Amount pumped. [4] Very little underdrainage; one line extends through certain beds at depth of 4 to 6 ft.; 40 ft. apart in one bed. [5] Three acres of filters under construction. [6] Includes sludge beds. [7] Only 3 beds underdrained. [8] Amount treated by intermittent filters only, day sewage.

Partition banks should be made low and narrow, to economize area. A height of 18 in., with a width of 2 ft. on top will generally be found sufficient.

UNDERDRAINS

The underdrains should be laid true to line and grade. The bottom of the bed, when artificially constructed, should be level or sloping toward the lines of underdrains. Where the beds are artificial it is desirable to lay the underdrains with surrounding gravel in trenches below the bottom of the sand, thus making the entire depth of sand available for useful work and avoiding drains or gravel coming too near the surface. It is usually advantageous to provide a free outlet, although submerged outlets will operate satisfactorily, and trapped outlets may be advantageous under certain circumstances.

Materials for Underdrains.—Vitrified salt-glazed sewer pipe appears to be most satisfactory for underdrains. It is easily cleaned and



Mold for Placing Material.

Filling around Underdrains.

FIG. 176.—Grading material around underdrains of sand filters.

durable. Cement pipe has not proved durable in some cases, apparently being attacked by acids formed in the beds. Blind drains are not desirable on account of the difficulty of ridding them of deposit and organic growths and because they do not afford a means for the rapid escape of the effluent.

Method of Laying Underdrains.—In laying the underdrain pipe, the spigot end of the pipe should be separated from the shoulder in the bell by a distance of about ⅜ in. to permit the ready flow of water through the joint into the pipe. The authors have found it advantageous to break the upper part of the bell off the pipe, leaving the lower portion, however, to assist in maintaining the alignment of the pipe, as shown by Fig. 176. The underdrain is then surrounded with screened gravel or broken stone of different grades, to prevent the sand of the bed from breaking through and silting up the pipe. Two or three grades of gravel are used, obtained by sieving it on at least 2 screens, the first

having about 1-in. mesh, the second about ¼-in. mesh. This gives 3 different grades of material. It is desirable to discard the stones coarser than 2½ in. in diameter or thereabouts. In placing this gravel the drain pipe is surrounded by a layer of the coarsest material, 3 in. in thickness, which is covered by a 3-in. layer of the next grade, and unless the sand of the bed is very coarse, a third layer of the third or finest material is required. The sand of the bed may then be placed upon the underdrain without fear of its breaking through.

In order that the sand may not wash laterally into the underdrains, the layers of gravel should surround the pipe and not merely cover it.

For the purpose of placing the gravel with certainty and rapidity, the authors have found the device illustrated by Fig. 176 very satisfactory. Its cost is insignificant.

Underdrain Spacing, Depth, and Size.—No rule can be given for the depth and spacing of underdrains. They are usually laid at a depth of from 3 to 4 ft. at the upper end, on a flat grade, 6 in. or more in 100 ft. The spacing of the underdrains will be determined by the effective size and depth of the material and the shape of the bed. Ordinarily an interval of 40 ft. has been found satisfactory, though with sand having an effective size of 0.08 to 0.12 mm. it will be found advantageous to decrease this interval to about 30 ft.

TABLE 148.—UNDERDRAINAGE OF INTERMITTENT FILTERS

Place	Distance between parallel lines of under-drains, feet	Diameter of under-drains, inches	Depth of trench, feet	Cost per acre for lateral under-drains	Cost per foot	Effective size of sand
Andover, Mass.	20	. . . .	5	. . . .	. . . .	0.15
Brockton, Mass.	60[4]	5 to 8	7.5 to 10	$300	$0.40–0.65	0.04–0.31
Clinton, Mass.	50	6 to 8	5 to 7	. . . . 280[1]	0.53–0.55[3] 0.35[1]	Very coarse
Hudson	100	5 to 8	5 to 6	70[2]	0.15[2]	0.10–0.60
Marlboro, Mass.[7]	30–40	5	5 to 8	1000	0.74	0.10
Natick, Mass.	35	3	4	. . . .	. . . .	0.16
North Attleboro, Mass.	53	4 to 8	5 to 6	215[8]	0.27[2]	Variable
Norwood, Mass.	34–47	6 to 8	5	250[2]	0.21[2]–0.30[2]	0.21–0.57
Pittsfield, Mass.	35[5]	3	4	. . . .	. . . .	0.15
Saratoga, N. Y.	225[6]	6 to 8	6.5	. . . .	. . . .	0.20
Worcester, Mass.	35–50	6 to 8	4 to 6	420	0.42	0.20–0.28

[1] Cost of work done in 1908 and 1909. [2] Contract price. [3] Contract price on original work. [4] No underdrains in porous beds. [5] Lateral drains connect with main drain through center of each bed. [6] One underdrain to each bed. [7] Beds built 1908–1911. [8] Not including pipe.

The size and slope of the underdrains will, of course, be determined by the amount of liquid to be handled and the area of the beds contributing to them. A diameter of 4 in. may be considered the minimum;

in some cases, particularly where deep deposits of sand and gravel are found, no underdrains are necessary. Some examples of underdrainage are given in Table 148.

Opinion is somewhat divided as to the desirability of terminating the underdrains in a riser coming to the surface of the ground for purposes of aeration and inspection. Where this is done care must be taken to protect the pipe against breakage and admission of sewage. They were so troublesome at Worcester that they were removed.

Main Collector Underdrains.—The underdrainage pipe should be connected with a main arterial system of underdrains, which may be laid either with tight or with open joints, the size of the pipe depending upon the quantity of water to be handled, the minimum diameter usually being 8 in. The outfall into the stream or open channel may advantageously be provided with a headwall of masonry. If the lines of the underdrains themselves are long, manholes or lampholes may be desirable at convenient intervals and provision should be made for sampling the effluent from each bed in small plants and each group of beds in larger ones.

Clogging of Underdrains.—In some sewage disposal plants handling large quantities of iron wastes, as at Worcester, considerable difficulty has been caused by the clogging of the underdrains. The iron in the applied sewage is mostly in the form of ferrous sulphate and is partly oxidized by the filter to the ferric state, red ferric hydrate often being precipitated in comparatively large quantities. This precipitation is most active as the sewage trickles from the sand into the gravel surrounding the underdrains, which usually contains an ample supply of oxygen. The ferric hydrate thus precipitated gradually fills the gravel surrounding the pipes and clogs the open joints, so that after about 5 years it was necessary to take up the drains, removing the gravel and relaying the drains with clean stone. Similar clogging may take place where ordinary city sewage is applied to the filter, for there is always some iron in the sewage and usually also some in the sand, which may be in the ferrous state and be changed into ferric hydrate by oxidation in the filter.

The record of the clogging of the underdrains of the Worcester filters, as given in the annual reports of the Superintendent of Sewers, is as follows:

In 1899, the first beds were put in service, and in 1904 the underdrains were found to be clogged at the joints. The pipes were taken up in 10 beds, the bells were knocked off around half the circumference and the pipes were relaid, the half of the barrel to which the bell remained attached being placed at the bottom. The gravel placed about the joints was graded and the finest part placed farthest from the pipe. There were 7800 ft. of underdrains relaid at a cost of 40.3 cts. per foot.

In 1907 trouble was experienced with the underdrains of the 11 other beds built prior to 1904, and the drains were taken up and relaid, as previously described, at a cost of 41.7 cts. per foot for 8000 ft. The next year 4200 ft. were relaid at an average cost of 35.7 cts. per foot. In 1909 the drains were relaid in 12 beds, constructed in 1904. The length was 14,300 ft. and the cost was 32.5 cts. per foot, exclusive of pipe which brought the total to about 42 cts. This work has been continued and experience indicates that the underdrains require relaying about every 5 years.

In a few of the larger plants in Massachusetts, the underdrains have become clogged by the passage of the sand into the gravel surrounding them. This clogging has been so serious in filters at Marlboro and Natick as to require the relaying of the underdrains. Drains have also become clogged in some instances by organic growths.

DISTRIBUTION OF SEWAGE

The control of the distribution pipe system depends upon the size of the filter plant and the topographic conditions. In a large plant the sewage may be applied to the beds in groups, the pipe system being controlled by master valves at certain central points, each point of discharge of the sewage upon a bed being further regulated by a gate upon the lateral. In small filter plants the distribution can most advantageously be controlled by operating the gates upon individual beds.

Size and Kind of Distribution Pipes.—The sewage distribution mains are usually built of vitrified salt-glazed pipe, laid with cement, sulphur, or other type of tight joint, to line and grade, like pipe sewers. The distribution pipe system should be figured on a liberal basis, in order to permit of the rapid application of doses to individual beds. Under some conditions it may prove more economical to operate the distribution system under a substantial pressure, when cast-iron distributors will be necessary, as vitrified pipe will not withstand the pressure.

Methods of Distribution.—The following methods of distribution have been successfully used:

1. Graduated troughs running nearly the full length of the bed, as shown in Figs. 177 and 178, the former showing the distribution and underdrainage of intermittent filters constructed at North Attleboro, Mass., from the plans of F. A. Barbour.

2. Fan-shaped or arterial troughs, used particularly for irregular-shaped beds.

3. Quarter-point distribution, which consists of the discharge of sewage at the 2 quarter points on the long sides of the bed.

4. Corner distribution, in which the sewage is discharged on to the

bed at or near its 4 corners, as shown in Figs. 179 and 180, illustrating the Marlboro, Mass., filters built from the authors' plans.

5. One or two middle distributors, located upon the long side of the bed as shown in Fig. 181. This method, which was used for a time at Clinton, Mass., has been abandoned there as unsatisfactory.

The discharge of sewage at these distribution points should be con-

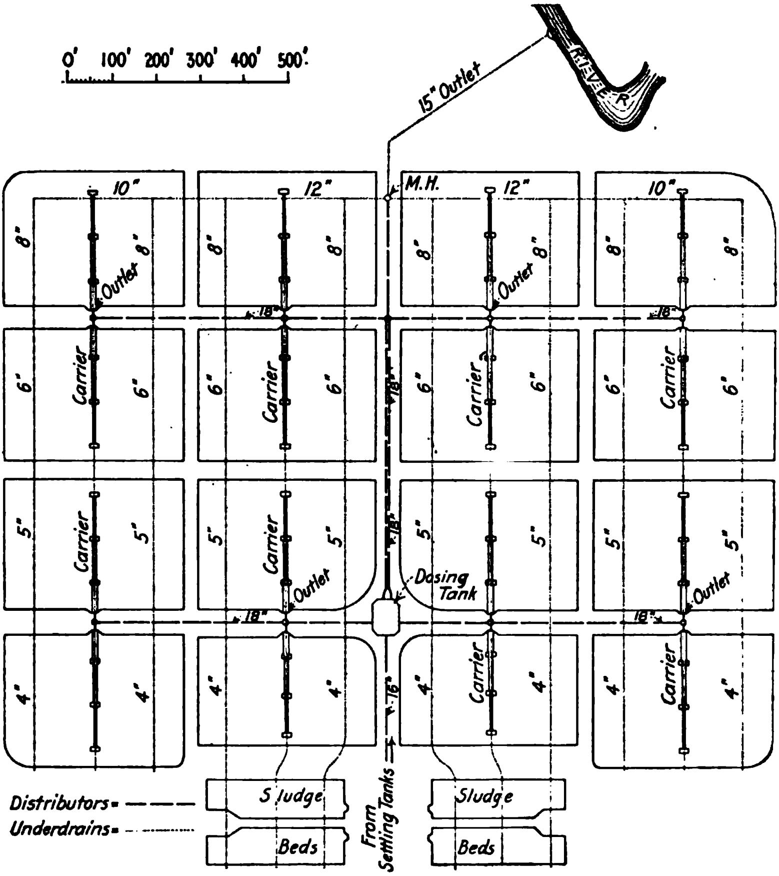


FIG. 177.—Arrangement of sand filter beds, North Attleboro, Mass.

trolled in a manhole, from which the distributor branches, by means of a shear or sluice gate. Of these the simple shear gate is the cheaper and has been found on the whole to give the more satisfactory service.

It is desirable to provide headwalls and a paved area at the point of discharge of the sewage, to prevent erosion of the surface of the bed, as shown in Figs. 178, 179 and 181.

FIG. 178.—Distribution trough of sand filter at Marlboro, Mass.

The quantity of sewage distributed along the line of the trough is fixed by moving the hinged wickets on the sides of the trough. Another view of this trough is shown in Fig. 22, page 231.

FIG. 179.—Outlet at sand filter, bed, Marlboro, Mass.

The sewage is under a head of 20 ft. at the end of the pipe and the block of concrete was placed as shown to break the rush of the sewage. The concrete and stone pavement prevents wash of the sand while the sheet of sewage is spreading out and losing its high initial velocity.

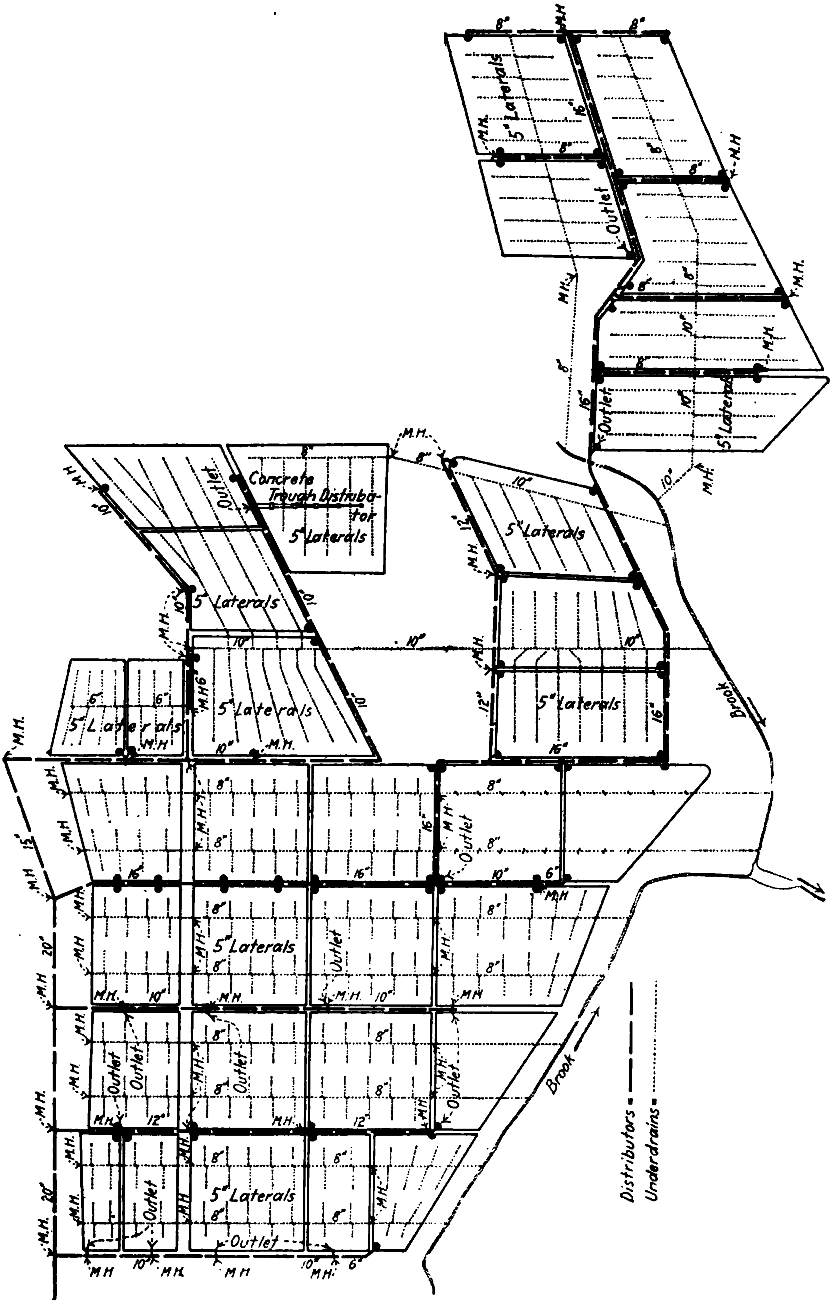


FIG. 180.—Arrangement of intermittent sand filters, Marlboro, Mass.

FRICTIONAL RESISTANCE OF FILTERS

The frictional resistance of sand to water has been studied in America by a number of investigators. The most useful formula for the determination of this resistance is that published by the Massachusetts State Board of Health in 1892; this was developed by Hazen.

Hazen's Formula.—In his "Filtration of Public Water Supplies," (1900, page 22), Hazen said:

"The frictional resistance of sand to water when closely packed,

FIG. 181.—Types of outlet used at Clinton, Templeton and Hudson, Mass.

with the pores completely filled with water and in the entire absence of clogging, was found to be expressed by the formula

$$v = cd^2 \frac{h}{l} \left(\frac{t + 10}{60}\right)$$

where v is the velocity of the water in meters daily in a solid column of the same area as that of the sand, or approximately in millions of gallons per acre daily.

c is an approximately constant factor.

d is the effective size of sand grain in millimeters.

h is the loss of head.

l is the thickness of sand through which the water passes.

t is the temperature (F.).

TABLE 149.—SHOWING RATE IN MILLIONS OF GALLONS PER ACRE DAILY AT WHICH WATER WILL PASS THROUGH EVEN-GRAINED AND CLEAN SANDS OF THE STATED GRAIN SIZES AND WITH VARIOUS HEADS AT A TEMPERATURE OF 50°

(c = 1000)

$\frac{h}{l}$	Effective size in millimeters; 10 per cent. finer than							
	0.10	0.20	0.30	0.35	0.40	0.50	1.00	3.00
0.001	0.01	0.04	0.10	0.13	0.17	0.27	1.07	9.63
0.005	0.05	0.21	0.48	0.65	0.85	1.34	5.35	48.15
0.010	0.11	0.43	0.96	1.31	1.71	2.67	10.70	96.30
0.050	0.54	2.14	4.82	6.55	8.55	13.40	53.50	......
0.100	1.07	4.28	9.63	13.10	17.10	26.70	107.00	......
1.000	10.70	42.80	96.30	131.00	171.00	267.00	....	......

"The above table (Table 149) is computed with the value c taken as 1000, this being approximately the value deduced from the earliest experiments. More recent and extended data have shown that the value of c is not entirely constant, but depends upon the uniformity coefficient, upon the shape of the sand grains, upon their chemical composition, and upon the cleanness and closeness of packing of the sand. The value may be as high as 1200 for very uniform and perfectly clean sand, and may be as low as 400 for very closely packed sands containing a good deal of alumina or iron, and especially if they are not quite clean. The friction is usually less in new sand than in sand which has been in use for some years. In making computations of the frictional resistance of filters the average value of c may be taken at from 700 to 1000 for new sand, and from 500 to 700 for sand which has been in use for a number of years.

"The value of c decreases as the uniformity coefficient increases. With ordinary filter sands with uniformity coefficients of 3 or less, the differences are not great. With mixed sands having much higher uniformity coefficients, lower and less constant values of c are obtained, and the arrangement of the particles becomes a controlling factor in the increase in friction.

"The friction of the surface layer of a filter is often greater than that of all the sand below the surface. It must be separately computed and added to the resistances computed by the formula, as it depends largely upon other conditions than those controlling the resistance of the sand.

"While the value of c is thus not entirely constant, it can be estimated with approximate accuracy for various conditions, from a knowledge of the composition, condition, and cleanliness of the sand, and closeness of packing.

"The following table (Table 150) shows the quantity of water passing sands at different temperatures. This table was computed with temperature factors as given above, which were based upon experiments upon the flow of water through sands, checked by the coefficients obtained from experiments

with long capillary tubes entirely submerged in water of the required temperature.

TABLE 150.—RELATIVE QUANTITIES OF WATER PASSING AT DIFFERENT TEMPERATURES FAHRENHEIT

32°........0.70	44°........0.90	56°........1.10	68°........1.30
35°........0.75	47°........0.95	59°........1.15	71°........1.35
38°........0.80	50°........1.00	62°........1.20	74°........1.40
41°........0.85	53°........1.05	65°........1.25	77°........1.45

"The effect of temperature upon the passage of water through sands and soils has been further discussed by Prof. L. G. Carpenter, *Engineering News*, vol. xxxix, page 422. This article reviews briefly the literature of the subject, and refers at length to the formula of Poiseuille, published in the *Memoires des Savants Etrangers*, vol. xi, page 433 (1846). This formula, in which the quantity of water passing at 0.0°C., is taken as unity, is as follows:

$$\text{Temperature factor} = 1 + 0.033679t + 0.000221t^2$$

The results obtained by this formula agree very closely with those given in the above table throughout the temperature range for which computations are most frequently required. At the higher and lower temperatures the divergencies are greater, as is shown in a communication in *Engineering News*, vol. xl, page 26.

"The quantity of water passing at a temperature of 50°F. is in many respects more convenient as a standard than the quantity passing at the freezing-point. Near the freezing-point, owing to molecular changes in the water, the changes in its action are rapid and the results are less certain, and, also, 50°F. is a much more convenient temperature for precise experiments than is the freezing-point."

Use of Hazen's Formula.—Owing to the frequent misapplication of this formula and failure to observe the conditions under which the Massachusetts experiments were made, particular attention is called to the following discussion by Hazen of a paper upon "Dams on Sand Foundations" by Koenig, *Trans. Am. Soc. C. E.*, vol. lxxiii, page 200:

"These results have been quoted many times, but in doing this their form has been sometimes changed, new assumptions have been introduced, and limitations originally made have been omitted, so that in using them at the present time, the only safe way is to refer to the original publication.

"There is no reason why flows through gravels should follow the formula given for sand, as the author seems to suppose. In the original publication it was clearly stated that this was not the case, as appears from the following quotations:

" 'The frictional resistance of sand to water within certain limits of size of grain and rate of flow varies directly as the rate and as the depth of sand. This is given by Piefke as Darcy's Law. I have found that the

friction also varies with the temperature, being twice as great at the freezing point as at summer heat both for coarse and fine sands, and also that with different sands the resistance varies inversely as the square of the effective size of the sand grain.'

"The formula was then given for the flow of water in sands with effective size between 0.1 and 3.0 mm. It was then stated:

" 'For gravels with effective sizes above 3 mm. the friction varies in such a way as to make the application of a general formula very difficult. As the size increases beyond this point, the velocity with a given head does not increase as rapidly as the square of the effective size; and with coarse gravels the velocity varies as the square root of the head instead of directly with the head as in sands. The influence of temperature also becomes less marked with the coarse gravels.'

"The table (Table 151) was, therefore, prepared showing the experimental results for gravels, not covered by the sand formula. It is also to be noted that in the original publication results were uniformly expressed in terms of meters daily in a solid column of water of the same area as that of the sand.

TABLE 151.—RATE IN METERS PER DAY AT WHICH WATER WILL PASS THROUGH DIFFERENT GRAVELS, WITH VARIOUS HEADS

(Mass. State Board of Health Report for 1892, page 555)

$\frac{h}{l}$	Effective size in millimeters; 10 per cent. finer than									
	3	5	8	10	15	20	25	30	35	40
0.0005	3.5	10	20	30	50	80	110	150	200	250
0.001	7.0	21	41	58	100	148	205	275	370	450
0.002	14.0	40	78	110	190	275	370	480	590	710
0.004	27.0	77	150	208	350	480	610	740	870	1000
0.006	41.0	112	207	275	450	620	780	930	1090	1240
0.008	54.0	142	252	340	530	720	900	1090	1270	1450
0.010	67.0	173	300	385	610	830	1030	1220	1410	......
0.015	98.0	238	378	480	760	1030	1260	1480	......	......
0.020	127.0	300	467	580	890	1180	1470	......	......	......
0.030	185.0	400	615	750	1110	1450	......	......	......	......
0.050	280.0	560	885	1060	1490	......	......	......	......	......
0.100	495.0	930	1310	1550	......	......	......	......	......	......

h = loss of head; l = thickness of gravel bed through which water passes.

"As a matter of fact, the variation of c in the original formula,[1]

$$v = cd^2\frac{h}{l}(0.70 + 0.03t)$$

is not especially great. The coefficient rarely falls below 400, even for old and dirty sand, and rarely rises above 1200, and, in a majority of ordinary sands, falls between 700 and 1000. This is certainly not a wide range.

"It is to be noted that the formula was never intended to apply to clays,

[1] In this form of the formula t is the temperature in Centigrade degrees.

hardpans, soils, and other materials. The effort to apply it to such materials is not to be encouraged, and the results are not to be depended upon.

"As an illustration of the practical use of this formula, take the case of the filter construction now under way at Toronto, Ont. A large permanent plant is being built on a sand island, cut by channels, with foundation some feet below the lake level. It was proposed to dig a drainage canal entirely around the site and pump the water out of it, thereby draining the whole site. Calculation made in August, 1908, assumed this drainage canal to be 3500 ft. long, and that the depth of material under the canal carrying water would be 50 ft., making a total area of 4 acres through which the water would flow. The sand was assumed to have an average effective size of 0.25 mm. and c was taken as 800. It was assumed that the average distance from the canal to the water outside would be 200 ft., and that the water in the canal would be 5 ft. below lake level, resulting in a ground-water slope to the canal of 0.025. At a temperature of 50° water would pass, by the formula, at the rate of 1.25 meters per day, or 1,110,000 imp. gal. per acre daily, and for the 4 acres of assumed cross-section of flow the total quantity of water would be 4,440,000 imp. gal. per day. The writer does not think that any of the practical men who looked at the site before it was drained believed that the removal of so small a quantity of water would suffice. As a matter of fact, the actual quantity of water pumped has never exceeded the computed quantity, and has ordinarily been only one-half or one-third as much, owing in part to the fact that the sand proved to be finer than was assumed (actually about 0.21 mm. on an average), and that the average distance from the canal to open water was greater and the hydraulic slope less steep than assumed."

Table 152 has been prepared to show the effect of different assumptions as to c on the discharge of a sand filter. For other ratios of h/l than 1, the tabular numbers should be multiplied by the assumed ratios. Temperature corrections can be made by means of Table 150.

The Hazen formula finds its greatest use in testing washed sand or sand which is clean and uniform. Even in careful hands it must be used with great attention to detail, except with very clean sand. For instance, in tests made for the Baltimore Sewerage Commission, described by Chief Engineer Hendrick in his report for 1906, washed sand with an effective size of 0.322 and a uniformity coefficient of 1.69, containing 41.4 per cent. of voids[1] when packed dry, had a coefficient c ranging apparently from 266 to 454. The average was 320. Another experimental filter with sand having an effective size of 0.231 mm., a uniformity coefficient of 2.51 and a porosity of 43.1 per cent. when packed dry, showed values of c ranging from 415 to 636, averaging 498. The sand in both cases had been washed by filling two-thirds of the depth of a bucket with the sand, turning a small stream of water into it,

[1] Experiments by Baldwin-Wiseman showed that by shaking loose sand of the grades used in a number of British water filters the volume of the sand was reduced from 12 to 29 per cent. (*Proc. Inst. C. E.*, vol. clxxi, page 60.)

and stirring the mass until the water running from the bucket appeared to be clear.

TABLE 152.—DISCHARGE OF SAND FILTERS IN MILLIONS OF GALLONS PER ACRE WITH $h/l = 1$ AT A TEMPERATURE OF 50°F., BY HAZEN'S FORMULA

Effective size, mm.	Value of c								
	400	500	600	700	800	900	1000	1100	1200
0.10	4.3	5.3	6.4	7.5	8.5	9.6	10.7	11.8	12.8
0.12	6.2	7.7	9.2	10.8	12.3	13.9	15.4	16.9	18.5
0.14	8.4	10.5	12.6	14.7	16.8	18.9	20.9	23.0	25.1
0.15	9.6	12.0	14.4	16.8	19.2	21.6	24.0	26.5	28.9
0.16	10.9	13.7	16.4	19.2	21.9	24.6	27.4	30.1	32.8
0.18	14.0	17.3	20.8	24.2	27.7	31.2	34.6	38.1	41.6
0.20	17.1	21.4	25.7	29.9	34.2	38.5	42.8	47.0	51.3
0.22	20.7	25.9	31.0	36.2	41.4	46.6	51.7	56.9	62.1
0.24	24.6	30.8	36.9	43.1	49.3	55.4	61.6	67.7	73.9
0.25	26.7	33.4	40.1	46.8	53.4	60.1	66.3	73.5	80.2
0.26	28.9	36.1	43.4	50.6	57.8	65.0	72.3	79.5	86.7
0.28	33.5	41.9	50.3	58.7	67.0	75.4	83.8	92.2	105.9
0.30	38.5	48.1	57.7	67.3	77.0	86.6	96.2	105.8	115.5
0.32	43.8	54.7	65.7	76.6	87.6	98.5	109.5	120.4	131.4
0.34	59.4	61.8	74.1	86.5	98.9	111.2	123.6	135.9	148.3
0.35	52.4	65.5	78.6	91.7	104.8	117.9	131.0	144.1	157.1
0.36	55.4	69.3	83.1	97.0	110.8	124.7	138.5	152.4	166.3
0.38	61.7	77.2	92.6	108.1	123.5	138.9	154.4	169.8	185.2
0.40	68.4	85.5	102.6	119.7	136.8	153.9	171.0	188.1	205.3
0.42	75.4	94.3	113.1	132.0	150.9	169.7	188.6	207.4	226.3
0.44	83.0	103.7	124.5	145.2	166.0	186.7	207.4	228.2	248.9
0.45	86.6	108.2	129.9	151.5	173.2	194.8	216.5	238.1	259.8
0.46	90.5	113.1	135.7	158.3	181.0	203.6	226.2	248.8	271.5
0.48	98.5	123.2	147.8	172.4	197.0	221.7	246.3	270.9	295.6
0.50	106.9	133.6	160.4	187.1	213.8	240.5	267.3	294.0	320.7

Three tests of the same sand unwashed were also made. A sample having an effective size of 0.075 mm., a uniformity coefficient of 5 and a porosity of 34.1 per cent., had a coefficient c of 32. Another sample with an effective size of 0.033, a uniformity coefficient of 11.5, and a

porosity of 11.5, had a value for c of 2. The third sample, with an effective size of 0.162, a uniformity coefficient of 2.17 and a porosity of 39.6 per cent., had a coefficient c of 8.

Mechanical analyses of sand containing large amounts of clay were made by the method of screening which was described on page 628, and in the report referred to the usual method of making such an analysis was termed "practically worthless" for sand containing as much clay as those examined. The filter tests showed the sand to be of as trifling value for sewage treatment as the mechanical analyses indicated.

OPERATION OF SEWAGE FILTER BEDS

Dosing.—It is essential in all processes of biological filtration, for reasons explained in Chapters III and VI, that an abundant supply of oxygen be admitted to all portions of the filter. For this reason the dosing and resting of the filter are of vital importance. It is found desirable to apply the sewage to sand filters in intermittent doses, in order to ensure proper aeration of the bed and good distribution of the sewage upon it.

The doses may be regulated by a dosing tank (see Chapter XVIII) or similar device, or in the case of large plants, by manual control of gates on the distributing system.

The amount and size of the dose depend largely upon the effective size of the sand, condition of the bed, and character of the sewage. From 1 to 3 doses may be successfully applied per day, but in practice it has ordinarily been found advantageous to apply not over one, proportioning the amount so as to give the filter ample time to recuperate through the admission of air after draining off. In some cases it has been found better to dose the filter with twice the amount of sewage on alternate days thus giving the filter a much longer time to aerate, but also requiring it to be in service longer. In some places the dosing has been more infrequent than this, ranging from 1 dose in 3 days to 1 dose at long and irregular intervals.

Clogging of Beds.—On beginning the use of a sewage filter bed the surface should be smooth and level. For the first few days of use uniform distribution of the sewage over the entire bed will not be obtained, for so much sewage will penetrate the sand near the distributor outlets that the dose will be exhausted before it has an opportunity to cover the bed. Gradually the surface will become partially clogged, with marked improvement in uniformity of distribution.

The degree and nature of the clogging vary with the character of the suspended matter in the sewage applied, as discussed in detail by Eddy and Fales. (*Jour. Assoc. Eng. Socs.*, vol. xxxvii, page 67.) If it contains a large quantity of coarse material, such substances will settle and

accumulate on the surface of the sand, the coarsest in the immediate vicinity of the distributor outlet and the finest at the farthest points from such outlet. In other words, the material will be more or less gradually distributed according to the size of the particles. With the lapse of time a surface mat of fibrous material will form, if raw sewage is applied. When allowed to dry out between doses, the mat separates from the surface of the sand and usually cracks and curls up, Figs. 182 and 183, the rapidity of drying depending upon wind and weather conditions. Such a mat is beneficial in preventing the entrance of very fine substances into the body of the bed. On the other hand, it must be removed at frequent intervals to enable the bed to receive its

Fig. 182.—Surface of filters dosed with unsettled sewage, Worcester, Mass.

proper quantity of sewage and the requisite air. If the coarse suspended matter could be uniformly distributed over the surface of the bed, the advantage of the mat in preventing the entrance of fine material into the sand might be sufficient to offset the cost and delay due to frequent cleaning, and might make it unnecessary to remove as much sand as when the sewage is settled before being applied to the filter. This is not usually the case, however, and sewage is commonly passed through sedimentation tanks before being discharged onto sand filters.

The suspended matter in settled sewage, although somewhat fibrous, is so finely divided that it penetrates the sand more deeply than the coarser matters present in raw sewage. The mat formed by the finer particles is not strong and heavy and is not effective in preventing more

FIG. 183.—Mat formed by suspended solids on surface of intermittent filters dosed with unsettled sewage, Worcester, Mass.

FIG. 184.—Partially cleaned surface of intermittent filter dosed with chemical effluent, Worcester, Mass.

finely divided matter from entering the bed. The suspended matter in the effluents from the septic tank and from chemical precipitation is still more finely divided and less fibrous than that from plain sedimentation. Consequently, such matters penetrate the sand to an even greater depth than the suspended matter from sedimentation. The suspended matter in these effluents does not form a mat and, therefore, it is necessary, in order to remove it, to scrape off a substantial portion of the surface sand. This is illustrated by Fig. 184, showing the partially scraped surface of a bed which had been treating chemical effluent.

The very finely divided silt washed from street surfaces in times of storm, if applied to sand filters, will cause serious surface clogging. Filters which are in good condition may be so clogged by the storm water from a single brief shower as to require scraping before they can be put into operation again.

Rain has the effect of beating down the surface of the filters, which may require raking before they will resume their normal capacity. On the other hand, frost has a tendency to open the pores of the bed, enabling it to receive sewage much more freely than it did just prior to freezing. Beds which are considerably clogged may be made so porous by freezing that their capacity will be temporarily much increased, providing the frost does not go deeper than an inch or two.

In spite of surface cleaning, the upper layers of sand store up organic matter which is gradually oxidized by bacterial action, but a certain quantity of humus remains which must eventually be removed. The chemical composition of some surface accumulations and of the mat from raw sewage solids is given in Table 153.

TABLE 153.—RESULTS OF ANALYSES OF MATERIAL SCRAPED FROM INTERMITTENT FILTERS
(Percentages)

	Brockton	Worcester, dosing with 3 different sewages			
		Chemical effluent		Settled sewage	Unsettled sewage (mat)
		Sand over drains	Sand between drains		
Moisture	16.22	15.3	11.5	22.87	6.13
Loss on ignition	....	11.9	6.8	....	51.45
Organic nitrogen	1.45	0.709	0.325	0.539	2.31
Calcium oxide	0.30	2.2	1.9	....	....
Ferric oxide	....	3.5	3.0	2.58	....

The operation of intermittent filters for many years at the Lawrence Experiment Station demonstrates that when the beds are dosed at rates enabling them to carry the process of nitrification to practical comple-

tion, the organic humus-like material accumulates in the sand very slowly. In the report of the Massachusetts State Board of Health for 1910, page 245, it was stated that filter 1, of sand 0.48 mm. effective size, after 23 years' operation at a rate of about 57,000 gal. per acre per day, with sewage which contained about 2.5 lb. of organic matter in every 1000 gal. (300 parts per 1,000,000), had successfully cared for, without retention, 96.5 per cent. of the applied nitrogen and 87 per cent. of the total applied organic matter. With regard to the two classifications of organic matter, the Board's report of 1908 contained the following statement by Clark and Gage:

"Nitrogen forms but a small percentage of the organic matter present in sewage and stored in filters. The reason that it has so prominent a place in all studies of water and sewage, and especially of sewage purification, is that it is the chief constituent of matter that is easily changed; matter which causes odors when putrefaction occurs, and matter the change of which from one form to another denotes the transformation of a polluted liquid, such as sewage, to a well-purified filter effluent. As a matter of fact, however, the clogging which occurs in sand filters is due chiefly to carbonaceous matter,,fatty and otherwise (page 328).

"The stored organic matters in the filters contain but comparatively little fat, none of the samples examined from these filters having more than 7.5 per cent. as much fat as carbon, the average being about 5 per cent. of the carbon present. There is a very large amount of fatty matter in sludge; it varies from 29 to 56 per cent. in the total organic matter in the samples of sludge examined" (page 331).

The experience at the Lawrence Experiment Station shows that with beds of suitable sand and moderate rates of operation, the storage of organic matter in sand will not be a serious matter for many years. Surface clogging will occur, of course, and top sand must be removed from time to time, even when the sewage is not applied more rapidly than the biological life in the bed can care for the organic matter reaching it. There is a certain part of the applied sewage which is as stable as the organic matter of soil and resists for long periods the changes due to chemical and biological forces. Preliminary treatments which take out suspended matter are particularly helpful in preparing sewage for intermittent filtration, because the removal of sand has been found to be directly proportional to the amount of organic matter applied, especially those portions in colloidal form and that in suspension. On this topic, Clark and Gage reported in 1908:

"It is noticeable that the removal of suspended organic matter from the strained, clarified and septic sewage was of great help in maintaining high rates with the sand filters receiving sewage treated in these various ways, but that, in order to maintain the rates, even though much of the matter in suspension was removed, more frequent and greater sand removal was

necessary. Furthermore the number of cubic yards removed per 100 lb. of nitrogen in suspension applied was from two and one-half to four and one-half times as great with the high-rate filters as with low-rate filter No. 1, the chief benefit obtained, of course, being the purification of a large volume of sewage upon a small area." (Rept. Mass. Bd. Health, 1908, page 333.)

The condition of sand at different depths in intermittent filters after several years is shown in Tables 154 and 155, giving the results of ex-

TABLE 154.—RESULTS OF ANALYSES OF SAND TAKEN AT DIFFERENT DEPTHS FROM WORCESTER, MASS., FILTER DOSED WITH CHEMICAL EFFLUENT

(Percentages)

Depth of sample, inches	Samples taken over drain			Samples taken between drains		
	Ferric oxide	Organic nitrogen	Calcium oxide	Ferric oxide	Organic nitrogen	Calcium oxide
Top– 2	2.50	0.146	0.545	1.08	0.089	0.106
2– 6	2.00	0.060	0.161	1.64	0.052	0.087
6–12	1.68	0.022	0.093	1.47	0.031	0.066
12–24	1.50	0.013	....	1.33	0.018	....
24–36	1.31	0.013	....	1.21	0.020	....
36–48	1.40	0.016	....	1.16	0.014	....
48–60	1.36	0.015	....	1.22	0.007	....
60–72	1.41	0.013	....	1.41	0.008	....

TABLE 155.—RESULTS OF ANALYSES OF SAND FROM WORCESTER, MASS., FILTER RECEIVING UNSETTLED SEWAGE

(Percentages)

	Depth, inches	Moisture	Loss on ignition	Organic nitrogen	Iron in terms of ferric oxide	Total sulphur	Sulphates in terms of sulphur
Samples over drains	Top– 6	5.56	0.89	0.040	1.42	0.0145	0.0097
	6–12	6.77	1.01	0.0246	1.65	0.0077	0.0077
	12–24	5.88	0.71	0.0163	1.53	0.0075	0.0063
	24–36	11.93	0.67	0.0145	1.33	0.0042	0.0042
	36–48	7.49	0.59	0.0082	1.36	0.0047	0.0047
	48–60	10.42	0.55	0.010	1.38	0.0047	0.0047
Samples between drains	Top– 6	4.48	1.13	0.039	1.38	0.0044	....
	6–12	5.03	0.87	0.021	1.22	0.0022	....
	12–24	4.41	0.68	0.008	1.31	0.0018	....
	24–36	4.83	0.65	0.008	1.17	0.0012	....
	36–48	6.35	0.73	0.008	1.20	0.0022	....
	48–60	8.20	0.66	0.007	1.35	0.0030	....

aminations made in 1905 at the sewage treatment works in Worcester, Mass., where the beds are worked to their full capacity all the time. The same conditions, considerably more emphasized, were encountered about the same time at Pawtucket, R. I., where the pores of the sand became heavily charged with organic matter. This experience convinced City Engineer George A. Carpenter that the beds should not be dosed at an average rate exceeding 50,000 to 60,000 gal. per acre per day for 365 days in the year. (*Jour. Assn. Eng. Socs.*, 1906, vol. xxxvii, page 100.) The condition of the Pawtucket bed, as indicated by the parts per 1,000,000 of albuminoid ammonia in the sand at different depths, was as follows:

Depth, inches	Top–3	3–6	6–12	12–24	24–36
Alb. ammonia:					
Original sand	5–16	5–16	5–16	5–16	5–16
Filter sand	350–390	260–440	120–210	50–90	20–90

Objections to Disturbing Surface of Filters.—Harrowing and plowing, which will be described later, may, under some circumstances, be absolutely necessary. There are, however, serious objections to this treatment of the filter, for the disturbing of the upper layers of the sand mixes the organic matter with the cleaner sand below. This organic matter is decomposed more or less, but it does not entirely disappear; on the contrary it changes to a sort of humus, which remains in the sand and has the effect of reducing its capacity. It is obvious, therefore, that a time will come in the life of the filter when this material will reach such an amount as to practically clog the upper layer of sand and require its removal. It is undesirable to cause the organic matter to penetrate the filter any deeper than is absolutely necessary, because this necessitates the removal of a greater depth of sand.

It is perhaps impossible to fix any definite rule for the manipulation of the surface of the filter, as there are doubtless cases where it would be as unwise to disturb the sand as it would be impossible to avoid it in others. The general principle, however, seems to be that care should be taken to avoid mixing the solids deposited by the sewage with the sand and, therefore, the beds should be thoroughly cleaned before harrowing or plowing.

Removal of Dirty Sand.—The surface mat should be removed at frequent intervals and as the proportion of organic matter to sand in the upper layers increases, it is also necessary to remove a certain amount of this layer of sand, humus and other clogging material. The amount of this material to be removed depends upon the character of sewage which has been applied to the filter, and probably also upon the manipulation of the surface of the filter. But the chief factor appears to be the rate at which the filter is operated. If the load is light the natural biological processes will oxidize the organic matter nearly as fast as it

is applied, but if the filter is forced such substances, more or less changed, will accumulate rapidly.

Disposal of Rakings.—The mat which is peeled from the surface of the filter and the material which is scraped in a moist condition from the sand contain a large amount of organic matter. The amount of nitrogen runs, as already shown, as high as 1.3 per cent. of the dried sample. This material has some value as a fertilizer, and may in some cases be entirely removed by local farmers, and if the quantity is not too great it is possible that a small revenue may be obtained from this source. The dirty sand which is removed contains a comparatively small proportion of nitrogenous matter, and seems to have no practical use. It must, therefore, be wasted at the least possible expense. Under some conditions it may be wise to wash this material, and replace it on the surface of the filters. This, however, is open to the objection that the finer particles will be removed, leaving a coarser-grain sand for the surface, which will allow the finely divided matter to pass through it, producing the effect of stratification.

Resting.—When the bed is overtaxed or the aeration becomes inadequate, fouling begins at the bottom of the bed, on account of the lack of air. Such a condition should be avoided by careful attention to dosing and care of surface, but if it exists the bed must be thrown out of commission, and allowed to dry out so as to overcome the effect of capillarity and become charged with air. Bacterial activity will still continue for a considerable period of time under such conditions, so that the period of resting may vary within wide limits without injury. If the bed is in very bad condition a period of 4 to 8 weeks may be required for its recuperation, while 1 or 2 weeks may suffice to cure less serious cases.

An example of the effect of this resting period was afforded by the Hudson, Mass., filter beds, after they had been overdosed with sewage containing a large admixture of wool-scouring waste. Several acres of beds became substantially useless before a plant could be built for treating the wool scourings separately. The clogged beds were allowed to lie fallow during the winter, and early the following summer, when they were again put into commission, they were found to have substantially recovered their filtering capacity.

Harrowing.—After the removal from the filter of the mat and dirty surface sand, it is frequently found that the surface of the bed requires some further treatment. This is especially the case after a severe rainstorm, when the surface of the sand seems to be packed and is too hard to admit readily the usual quantity of sewage, and when the surface sand contains a large quantity of finely divided organic matter.

To meet these conditions it has been customary in some places to rake or harrow the filters to a depth of from ½ to 2 in. or more. This should never be done until the surface of the bed has been cleaned.

Where the area is small, the sand may be loosened by raking with an ordinary iron garden rake. Upon large areas, this work must be done by horse harrows, one of the most satisfactory being the Deere Universal steel smoothing harrow, of the Ames Implement & Seed Co., Boston, Mass. This harrow is made single or double for 1 or 2 horses. It has spike teeth which can be adjusted as to length, and the lever enables the operator to adjust the angle of all teeth at one time. The advantage of this type of harrow lies in the fact that it does not mix the top sand containing some organic matter with the clean material below as much as some other types.

Plowing.—When the beds are hard below the depth to which the harrow can reach, it may occasionally be necessary to plow them. This can best be done with a subsoil plow such as that known as the John Deere (Taylor) subsoil plow of the Ames Implement & Seed Co. With this plow it is possible to reach a depth of about 18 in. and there is little danger of mixing the top sand with the material below.

Replacing Sand.—So far as can be learned but few intermittent sewage filtration plants have been in operation a sufficient length of time to make necessary the replacing of material scraped off during the process of cleaning, although some industrial plants have required such renewal. It is quite probable that filters which are used well up toward their maximum capacity must have so much material removed that ultimate replacement will be absolutely necessary. Thus at the end of an 11-year period of use, the removal of 6 in. of material proved necessary at Brockton, Mass., at a cost of $4314 for 19 acres of beds. At Worcester it was found that on the beds taking effluent from chemical precipitation tanks, clogging due to change in the character of the surface material, caused by the depositing of lime, iron, and organic matter, necessitated the removal of about 4 in. of material 3 years after putting these filters into service. Experience showed that it was inadvisable to rake, break, or otherwise mix the surface material with that below it. After that time an average of about 3.3 cu. yd. of material per 1,000,000 gal. of chemical effluent filtered were removed. In the case of the effluent from the septic tanks, the quantity of material removed from the beds was substantially the same as when the chemical effluent was applied, but the clogging material had to be removed at more frequent intervals. With the effluent from settling tanks the amount of material requiring removal was somewhat less, though the total amount, including the mat, was about the same. The quantity of mat and dirty sand removed from Worcester beds receiving raw sewage has been as high as 14 cu. yd. per 1,000,000 gal. sewage filtered.

William S. Johnson has suggested that an allowance in maintenance should be made for the removal of 1 in. of sand per year from the surface of sewage filter beds.

MAINTENANCE IN WINTER

The warm, dry weather of the summer is by far the best period of the year for the operation of intermittent filters. The spring is wet, and the filters are then generally overcoming the ill effects of the previous cold weather, so that the best work is not done by them until the late spring or early summer. The fall is apt to be cold and rainy, and such weather is not advantageous. The winter months are the most difficult in which to get good results.

During cold weather the temperature of sewage varies from 40° to 55°F. (Table 37), and there is enough heat to thaw the frost in the bed and to keep it in operating condition. A moderate amount of frost in the sand has been found to open its pores, and thus to increase the amount of sewage which it can pass. It has frequently been observed at Worcester that filters which were working slowly during the fall would show increased capacity when the temperature was reduced sufficiently to freeze the sand to a depth of 2 or 3 in. If, however, the bed becomes frozen to a considerable depth it may be impossible to thaw the frost even with the largest dose of sewage which can be applied.

The success of the filter during winter weather depends somewhat upon chance. If the filters are in good condition, and a heavy snow storm occurs while the filter is flooded, the snow will be melted, and the water will readily find its way into the sand. On the other hand, if the filter is seriously clogged with suspended matter from sewage previously applied, and is very slow in filtering, the water will be chilled by the snow which falls into it. And if there should be a sharp decline in temperature it is quite probable that the snow and water will be frozen to a solid field of ice, which adheres tenaciously to the sand on to which it is frozen. If a considerable depth of snow falls when the filter is not in use, it has been found that when water is turned on it is immediately chilled, and in extreme weather it is quite likely to freeze and make the filter substantially impermeable. On the other hand, if the filter is in fairly good condition, and paths are shoveled through the snow giving the warm sewage a chance to get to the center or extreme points of the bed without becoming too cold, it has been found possible to thaw a large fall of snow and still keep the filter in good condition.

More care is required in dosing filters during the winter than at other seasons. Large doses of sewage must be applied to thaw the frost in the sand or to melt the snow on the surface of the filter. On the other hand, it is essential that the dose be not large enough to freeze to the sand before it can find its way into the filter.

Various methods have been devised for preventing the ice from freezing to the sand. One of the most successful of these is to furrow the filters for use in winter. The furrows may consist of ridges and depres-

sions about 3 ft. on centers and about 10 in. deep. When ice has formed on the sewage, the ridges hold it up as the sewage recedes, leaving a space between the bottom of the furrow and the ice, through which future applications of sewage can run. This covering of ice also protects the surface of the bed from the extreme cold, preventing serious freezing, and serves to keep succeeding snowfalls out of the sewage.

At Gloversville, N. Y., Harry J. Hanmer, City Engineer, states that the beds are furrowed with a double mold board plough made by H. H. Lovejoy, Cambridge, N. Y., which is particularly well adapted to this work. The ridges are made 2.5 ft. apart on centers. The furrows are 9 in. deep below the surface of the beds and the ridges are heaped up 9 in. above the surface making the furrows 18 in. deep. They are 6 in. wide at the bottom and no hand shaping or trimming is required.

This same result is obtained by means of hills or mounds, such as result from a cropping of the surface of the filter with corn, or by placing stones at frequent intervals. Boards on edge spiked to stakes driven into the filter have also been used for holding the ice up from the sand.

Furrowing and the making of hills, as in cropping, are open to the objection of mixing the suspended organic matter deposited by the sewage with the filtering material. It is, therefore, necessary to thoroughly clean the beds before they are furrowed in the fall and to clean the furrows in the spring before smoothing the bed. In some cases it is necessary to scrape up a considerable accumulation of dirty sand in the fall of the year. In this case, mounds may be made of this material and left on the beds during the winter. In this way the ice is as effectively held up as in the case of furrowing, and the sand is not disturbed. The mounds should be preferably not over 3 ft. apart and about 1 ft. high.

Winter Cleaning.—When sewage is applied to furrowed filters the suspended matter tends to settle in the bottom of the furrows, which afford a much smaller area for the distribution of such matter than the level area of the bed at other seasons. Where beds are operated at nearly their maximum capacity, especially with sewage containing large quantities of suspended matter, the winter clogging may be serious. It is important, therefore, to take advantage of any opportunity afforded for winter cleaning. In the course of nearly every winter in Central Massachusetts there is at least one period lasting for from one to several days, during which the temperature conditions are favorable for cleaning. If at such time the accumulation of ice does not prevent, it is desirable to remove as much of the clogging accumulation as possible. The work must be done expeditiously, however, as such periods are usually of short duration. It has been found that the mat can be readily raised from the sand if it is barely frozen. If, however, it is frozen hard, the crust will be so thick and the quantity of sand clinging to the mat so great, that cleaning will be impracticable.

Efficiency of Intermittent Sand Filters.—When sand filters are doing their best work the effluent from the underdrains will be clear, substantially free from suspended matter, and practically odorless. It will remain stable indefinitely even when allowed to stand in a tightly stoppered bottle in a warm room.

The efficiency of various sand filters in Massachusetts is indicated in Table 157. These results are for the most part the yearly averages of monthly samples. It will be seen that there is considerable difference in the quality of the sewages applied as well as of the effluents. It is probable that a majority of the analyses of applied sewages represent more accurately the stronger day sewage. Filtration does not affect the quantity of chlorine, so that the chlorine results may be used to determine whether the effluent probably corresponds to the sewage analyzed. It will be seen that there are wide discrepancies, and these should be taken into consideration when studying the results. In some cases, the effluent has doubtless been diluted with ground water. These

TABLE 156.—MONTHLY VARIATIONS IN EFFLUENTS OF INTERMITTENT FILTERS, BASED ON AN ANNUAL AVERAGE OF 100

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
	Free Ammonia											
Brockton..	100	163	192	171	134	92	79	63	50	50	50	83
Lawrence..	213	269	204	168	84	48	16	6	8	14	32	120
	Albuminoid Ammonia											
Brockton..	87	130	130	174	87	87	87	87	87	87	87	87
Lawrence..	175	171	150	132	92	76	67	58	62	49	62	117
	Nitrates											
Brockton..	85	70	67	81	90	112	118	115	133	129	110	96
Lawrence..	48	38	59	100	139	140	120	116	128	125	109	75
	Oxygen consumed											
Brockton..	84	132	152	178	100	84	72	68	68	84	92	108
Lawrence..	170	167	149	122	84	70	69	61	62	64	16	124

Lawrence figures are average of ratios of filters 1, 2, 3, 4, 6 and 10 (1895-1900).
Brockton figures (1897-1904).

TABLE 157.—EFFICIENCY OF SAND FILTERS IN MASSACHUSETTS*

Results of analyses in parts per 1,000,000

City or town	Free ammonia			Total albuminoid ammonia			Oxygen consumed			Nitrogen in eff. as		Chlorine	
	Applied sewage	Effluent	Per cent. reduced	Applied sewage	Effluent	Per cent. reduced	Applied sewage	Effluent	Per cent. reduced	Nitrates	Nitrites	Applied sewage	Effluent
Amherst	25.6	7.3	71	8.8	0.55	94	53.3	6.2	88	6.10	0.11	234.1	86.0
Andover	35.0	19.9	64	8.4	1.11	87	61.8	10.4	83	1.20	0.07	61.8	68.5
Brockton	67.7	31.8	53	13.3	1.12	92	128.9	15.3	88	4.07	0.12	122.3	120.3
Clinton	38.4	6.6	83	6.9	0.66	90	69.5	8.0	89	8.91	0.07	59.1	52.7
Concord	21.4	2.5	88	5.6	0.12	98	43.2	1.9	96	5.86	0.02	38.8	34.8
Framingham	61.1	17.4	72	15.0	0.70	95	111.3	9.1	92	2.48	0.12	94.9	84.0
Gardner[1]	54.0	14.6	73	17.6	1.01	94	136.1	8.0	94	19.04	0.19	61.6	63.2
Gardner[2]	43.8	21.6	51	7.4	1.82	76	46.2	13.2	71	9.19	0.23	74.8	67.6
Hopedale	49.0	13.1	73	7 1	1.16	84	44.0	9.2	79	21.30	0.06	58.3	50.0
Hudson	45.2	12.1	73	9.1	1.02	89	95.9	9.9	89	8.32	0.20	460.8	450.0
Leicester	35.5	6.7	81	7.1	0.98	86	56.7	9.5	83	6.07	0.13	54.5	40.6
Marion	12.2	2.0	84	3.3	0.37	89	21.8	3.9	82	4.58	0.02	37.6	36.5
Marlboro	51.8	5.0	90	5.7	0.36	94	44.5	3.8	91	18.47	0.04	98.6	78.6
Milford	39.2	6.9	82	6.1	0.39	92	51.8	5.1	90	15.04	0.09	98.7	97.9
Natick	36.7	11.6	68	8.5	0.57	93	57.8	5.9	90	4.76	0.21	89.0	65.7
Northbridge	37.7	6.5	83	6.5	1.07	84	43.1	7.5	83	9.23	0.08	54.2	43.8
Norwood	40.7	12.3	70	8.4	0.75	91	121.7	11.8	90	1.91	1.56	34.7	298.0
North Attleboro	11.5	0.6	95	1.8	0.12	94	12.8	2.0	84	4.94	0.02	34.1	29.9
Pittsfield	20.4	3.9	81	5.1	0.44	91	36.5	5.2	86	10.31	0.10	52.1	46.9
Southbridge	52.2	19.1	63	7.4	0.87	88	55.5	9.6	83	1.96	0.12	68.2	59.5
Spencer	56.8	5.4	91	12.6	0.38	97	72.3	5.1	93	4.77	0.06	88.3	55.0
Stockbridge	13.3	1.7	87	2.9	0.34	88	18.9	3.6	81	3.02	0.03	20.3	27.9
Westboro	31.4	4.1	87	8.4	0.72	91	49.7	7.0	86	11.52	0.28	82.9	48.8
Worcester	36.4	21.7	40	12.9	1.50	88	126.8	18.2	86	2.20	0.15	132.4	138.3

* Data taken from Massachusetts State Board of Health Report for 1912, pages 371–379. [1] Gardner area. [2] Templeton area.

results are not given with a view to showing the relative amount of work done at different plants, as other conditions must be taken into account in making such comparisons, particularly the character and quantity of sewage applied. These results do show, however, in a general way, the purification accomplished in practice.

Efficiency in Winter.—The efficiency of sewage filters in a northern climate will not be as great during the winter months as during the remainder of the year. Bacterial activity is reduced by fall in temperature. The large and irregular doses applied to the filters in order to keep the beds thawed out and to obtain the best results during the cold weather are not productive of a high degree of purification.

The relative efficiency of sand filter beds in summer and winter was investigated by Winslow and Phelps (U. S. Geol. Survey, Water Supply Paper No. 185). The monthly variations in the sand filter effluents at Brockton and at the experiment station in Lawrence, Mass., were found to be as stated in Table 156.

ODOR FROM SEWAGE FILTER BEDS

Opinions differ widely as to the odor coming from sewage filter beds. A characteristic sewage odor can be detected in their vicinity, particularly on wet days. The distance to which this odor is carried varies with conditions of wind and weather, as well as of the filter beds themselves. In some cases no objection has been raised from residents within 600 ft., more or less, of the filtering area, but it is probably desirable to keep the beds upward of one-quarter of a mile away from any substantial settlement, and the larger the area in use the more desirable is it that the beds should lie at a considerable distance.

The odor is not believed to have any serious effect upon health, for the men employed at the various sewage filter and broad irrigation plants are usually strong and healthy.

COST OF INTERMITTENT FILTRATION

Cost of Construction.—The cost of constructing filter beds will usually be found to lie between $2500 and $5000 per acre, although in some favored localities this cost may not exceed $1000. If the beds have to be built wholly artificially the cost may reach $10,000 per acre, if the sand has to be hauled a considerable distance.

Cost of Operation and Maintenance.—This cost will be found to lie ordinarily between $100 and $150 per acre, the cost per 1,000,000 gal. of sewage filtered being about $10, as will be seen from Table 158.

TABLE 158.—AVERAGE ANNUAL COST OF FILTRATION AREAS IN FOUR MASSACHUSETTS CITIES

(Compiled from Annual Reports)

City	Period	Population	Filter area acres	Mil. gal. filtered daily	Cost per acre	Cost per mil. gal.	Cost per capita
Brockton.....	1896–10	34,500–56,900	21.48–35.77	0.501–1.297	$178	$13.50	$0.10
Clinton......	1900–10	13,700–13,100	23.5–25.0	0.625–0.829	110	9.26	0.20
Concord	1901–10	5,600–6,400	3.3	0.273–0.264	108	3.33	0.06
Worcester[1]....	1904–10	129,500–146,000	3.3–74.3	0.273–4.718	290	10.37	......

[1] As only part of the sewage is treated by intermittent filtration, no satisfactory figures of the cost per capita can be given.

CHAPTER XVII

IRRIGATION AND THE AGRICULTURAL UTILIZATION OF SEWAGE AND SLUDGE

The general features of surface irrigation were outlined in Chapters I and VI, and it has already been pointed out that the popular opinion of the value of sewage in agriculture is much exaggerated. The fertilizing value of sewage is far less than is commonly supposed, on account of the great dilution of the constituents serviceable to plant life, nitrogen, phosphates and potash, and, further, because only a part of these substances is present in the sewage in a form suitable for fertilizing purposes. The nitrogen should be in the form of nitrates to be of most value, whereas usually only a small part, if any, of it has been brought to that stage and the remainder must be nitrified, which requires the presence of oxygen, mild temperatures and lime or some other base. Nitrification is checked if sewage is turned over land in too great quantities or if the air is cold, and if the sewage is applied freely there is a tendency to wash out of the soil what nitrates have formed.

In considering the fertilizing value of sewage it is also necessary to consider its ingredients which are detrimental to agriculture. The fat and soap may work harm by clogging the pores of the soil and thus counterbalance the small improvement due to the nitrogen, phosphoric acid and potash. One of the main objects of early chemical precipitation works in England was to remove such substances from the sewage that the partly clarified liquid might be less likely to cause the soil to become clogged and rank, a condition termed "sewage sick" by the managers of sewage farms. This precipitation removes some of the fertilizing elements of the raw sewage, particularly the suspended nitrogenous substances, but the soluble substances containing nitrogen, phosphates and potash remain.

In agriculture, fertilizers should be applied at certain stages in the rotation and growth of crops, and the proper fertilizers to use depend upon the nature of the soil, the climate, the crops to be grown and the rotation of crops. In sewage disposal, all these considerations must be waived in favor of the production of an effluent of suitable character. The crops should be regarded as merely a by-product. All evidence furnished by many years experience in many countries under many conditions does not reveal, however, any decisive proof that it is possible to obtain much fertilizing value from city sewage as it must be used to make

irrigation practicable, but indicates that where sewage irrigation has been successful agriculturally, irrigation with water would have produced about the same results. English experience indicates that whatever profit is to be made in the future from the fertilizing ingredients of sewage will probably result from the production of artificial manures from sludge.

Fertilizers.—The most important plant food is water, and most of the other plant foods, potash, lime, iron and silica, are furnished in solution in this water. Nitrogen, another important food, is largely taken up by the plants from solutions absorbed by the roots, but some leguminous plants absorb it from the air in the pores of the soil. It is evident, therefore, that solubility is an important index of the value of materials for fertilizers, and should determine how different fertilizers should be used. Barnyard manure, like the manurial constituents of municipal sewage, passes very slowly into solution and consequently it is applied in the autumn, so as to be slowly leaching for months before the fertilizing action is needed. Artificial ammonia and nitrate fertilizers, on the other hand, are dissolved so easily that they are not applied until just before they are needed.

The ingredients most needed in fertilizers are potash, phosphoric acid and nitrogen. Potash is usually obtained as muriates and sulphates, and all forms used in fertilizers are freely soluble. Phosphoric acid comes mainly from phosphates, in which it exists in combination with lime, iron or alumina. It occurs in three forms: 1, soluble in water and readily taken up by plants; 2, slightly soluble but nevertheless readily assimilated, in which condition it is termed reverted; 3, very slightly soluble and assimilated slowly. The first and second forms are collectively termed "available" phosphoric acid. The term "superphosphate" is applied commercially to any material containing soluble phosphoric acid as its chief constituent. Nitrogen is usually supplied as sulphate of ammonia or nitrate of soda.

TABLE 159.—FERTILIZING MATERIALS IN LAWRENCE SEWAGE

(Clark, Bulletin Mass. State Board of Health, December, 1913)

Ingredient	Parts per 1,000,000	Per 1000 gal.		
		Pounds	Cents per pound	Value, cents
Nitrogen as free ammonia	32.0	0.27	16.0	4.3
Kjeldahl nitrogen	10.8	0.09	10.0	0.9
Phosphorus	10.0	0.08	5.0	0.4
Potash	15.0	0.12	4.2	0.5
Total value of fertilizing ingredients	......	......	......	6.1
Each 1,000 gal. also contains fats worth	......	0.25	3.0	0.7

The ingredients in the sewage of Lawrence, Mass., possessing any fertilizing value were reported as in Table 159, by H. W. Clark. In a general way it is often stated that five-sixths of the ammonia which is capable of being generated from excreta is furnished by the urine, weich is more valuable as a fertilizer than the feces. Analyses of both classes of excreta are given in Table 22.

Opinions of Fertilizing Value of Sewage.—The loss of nitrogen during the purification of sewage was apparently considered of little practical importance by the Royal Commission on Sewage Disposal. This was because there was no known economical method of extracting the nitrate from a sewage effluent except by the agency of plants, and this agency cannot be employed on all occasions in an efficient way. The following statements were made in the Commission's Fifth Report:

"The most important manurial constituent of sewage is the ammonia which is produced by the fermentation of the urea of the urine, but it also contains organic nitrogen compounds in smaller quantity, together with phosphates and salts of potash. In the process of sewage purification by artificial filtration, varying quantities of the nitrogen of the ammonia and other compounds disappear, partly, no doubt, from conversion into gaseous nitrogen, and partly from being assimilated by vegetable growths, worms, flies, etc.; the remainder of the nitrogen is converted (in a well-purified effluent) into nitrate.

"It is in the form of nitrate that nitrogen is taken up by most plants. Much has been written about the loss to the country arising from the non-utilization of the nitrogen of sewage which has been purified by artificial filtration, a loss which is certainly to be deplored. It is, however, frequently forgotten that when water-borne sewage is purified on land, upon which grass or other crops are grown, loss of nitrogen is, to a large extent, unavoidable in the colder seasons of the year, since plants can only assimilate nitrate very slowly in cold weather. As regards the production of nitrate in the soil, it is well known that this is largely influenced by temperature, coming practically to a standstill when the temperature is very low, but it is probable, we think, that in this country the temperature would very rarely be sufficiently low for a long period, to arrest nitrification. During our observations on sewage farms, we had no experience of a very severe winter, but nitrification was not materially hindered by such cold as we did experience. It will, of course, be remembered in this connection, that the temperature of sewage itself is comparatively high" (page 150).

The opinion of the fertilizing value of sewage and sewage sludge held by Clark was stated by him in the Bulletin of the Massachusetts State Board of Health, as follows:

"The total amount of fertilizing and fatty matters in each 1,000 gal. of representative American sewage is not worth above 6 or 8 cts. Of this about half is represented by the ammonia in solution, and there is no method by which this material can be utilized except by application of the sewage

to land. All experience, covering many years, with hundreds of well-operated sewage farms ranging in size from a few acres to the vast 39,000-acre farm at Berlin, Germany, has shown that only under the most favorable conditions can the return from these farms be made to pay operating expenses, and an instance is yet to be cited where these returns pay both the cost of operation and interest on the capital invested. The exceptions, perhaps, to this are certain tracts or farms in regions of low rainfall and where the sewage is valuable as a liquid, that is, for real irrigating purposes.

"Much of the valuable fertilizing and fatty constituents in sewage is found in the matters in suspension. Average American sewage contains, perhaps, about 2400 lb. of sedimentable matters in 1,000,000 gal. and the nitrogen, fatty matters, etc., in this 2400 lb. of sludge are worth, approximately, $15 to $18. In order, however, to reclaim this valuable material, the sludge must be dried, pressed and also subjected to a process for the separation of grease from the fertilizing constituents in the remaining body of the sludge. Only by this separation can the grease become an article of commerce and the sludge of real agricultural value. This fact is well established by long experience and many investigations. When the fatty matters are separated by any known process, this procedure is costly. Only in a few places as yet is such separation attempted as a commercial enterprise, and the profitableness of the works at these places is yet doubtful. When the sludge is freed, or practically freed, from fatty matters it consists of a large weight of inert mineral and organic matter mixed with a comparatively small weight of fertilizing matter, and hence the cost of carriage is greater even when it is well dried. It has also been well proved that the nitrogen, phosphoric acid, etc., present are generally in a less assimilable form than the same bodies in ordinary commercial fertilizers. The sludge has value, however, and as the processes of drying, pressing and fat separation are improved, and also as nitrogen advances in price, as seems inevitable, sewage sludge will become of greater agricultural value than it is at present, especially as the basis of a fertilizer enriched by the addition of potash, phosphates, etc." (December, 1913.)

The fertilizing value of sewage sludge was summed up as follows by the New York Metropolitan Sewerage Commission, in its report of 1914:

"In a general way it may be said that under favorable conditions as to transportation, a sludge containing 50 per cent. moisture, whose dried material contains 3 per cent. of ammonia and less than 10 per cent. grease, might be further dried, ground and sold as a filler for fertilizer with some slight profit in the case of large works; but that no other than an occasional and uncertain offset to a part of the cost of operation can be looked for, even under favorable circumstances, from the sale of sludge in the form of crude cake or containing over 30 or 35 per cent. of moisture." (Report for 1914, page 403.)

Sludge as Fertilizer.—The fertilizing value of sewage sludge was investigated at the Woburn, England, experimental farm in 1907 and 1908 by Dr. J. A. Voelcker, whose conclusions from the results are given in the

Fifth Report of the Royal Commission on Sewage Disposal, Appendix 4. Some of the experiments were made by applying sludge from different works to grass land at the rate of 40 lb. of nitrogen per acre, and then weighing the hay produced per acre. While the method was admittedly crude, the results were considered to throw doubt on the fertilizing value of sludge and on the nitrogen content of the material being of importance, for the presence of lime was apparently much more important. It was suggested that the value of the lime might have been to render the nitrogenous organic matter available as plant food. During the same period, experiments were also carried out by T. H. Middleton for the Commission, who reported, in the same appendix, that any manurial value of sludge for grass land was not proved.

A somewhat different result was obtained by Dr. Voelcker after 2 years' work with grain fields. The sludge was applied at the rate of 40 lb. of nitrogen per acre, and increased the wheat crop of that year about 10 to 12 per cent. above the crops on land without manure. This increase was in both grain and straw, but was not so great as that obtained by using artificial manures supplying equal amounts of the same substances. Practically no manurial value was left in the soil after the first wheat crop. The sludges containing the most moisture and the most lime proved most effective, but $2.50 a ton was considered an outside value for any that was tested. Further experiments were carried out in 1914, using sludge and degreased sludge on land where hay and oats were raised. In the appendix to the Commission's Ninth Report it is stated that neither material had any appreciable effect, although mineral fertilizers increased the yield 22 to 47 per cent. above that of unfertilized land. In pot cultures, however, untreated and degreased sludge increased the yield of wheat 25 and 14 per cent. respectively. In both sets of experiments the degreased sludge appeared inferior to the natural material.

Experiments were also conducted for the Royal Commission on Sewage Disposal by a number of agricultural stations in 1906, using 7 different sewage sludges, various commercial fertilizers, and no fertilizing materials. The results were summed up as follows by T. H. Middleton in Appendix 8 to the Commission's Fifth Report:

"The only definite conclusion which may be stated is that, both for root crops and grass, the action of the nitrogenous and phosphatic constituents of sludge is very slow, as compared with the effect produced by nitrogen and phosphates supplied in ordinary artificial manures. For such crops as mangels (fodder beets), potatoes and swedes (coarse turnips), which have a short period of growth and require quick-acting manures, sewage sludges would not appear to be well adapted, and if they are employed they should be applied in tons rather than in hundred-weights per acre. On the other hand, although these experiments do not supply the evidence, it seems likely

that sludge used in proper quantities would form a good dressing for the slow-growing plants of many permanent pastures and meadows. Sludge is unlikely to give satisfaction on the very poor clay-soil pastures which are so much benefited by basic slag, but for old grass land of moderate good quality it should prove useful" (page 14).

In this connection, attention should be called to a statement made by Dr. A. C. Houston regarding the bacterial value of sewage sludges. Nitrifying bacteria play an important part in scientific agriculture, and the claim has been made that sludge would prove a valuable fertilizer because of the large number of bacteria present in it. Dr. Houston stated, in the same appendix to the Commission's Report (page 11), that unless the presence of multiple spores of bacteria can be considered as paving the way toward the decomposition of the sludge and setting free substances suitable for plant life, the physical characters and chemical composition of the sludges are the determining factors in their agricultural value.

The subject has been investigated by Lipman and Burgess of the University of California in connection with comprehensive researches into the changes to which the organic nitrogen of various commercial fertilizers is subjected. These were carried out with nine sludges containing the fertilizing values given in Table 160. The commercial value of such dry material does not exceed $10 a ton, they report, even on a liberal basis of the conventional calculations made by fertilizer chemists. The "availability" of the nitrogen, by which they consider such materials should be judged, was determined by a method, originated by them, which they claim gives reliable comparative results and probably accurate quantitative results. One gram of thoroughly dry, finely ground sludge was mixed with 100 grams of soil, and enough water added to produce the best moisture conditions. The tumbler containing this sample was covered with a Petri dish cover and incubated for a month at 28° to 30°C. At the end of this period nitrate determinations were made on these soil cultures, 6 different soils being used.

It was found that not only do different sludges behave differently in any one soil, but the different soils manifest markedly different capacities for rendering the nitrogen of sludge "available" in the general sense. Furthermore it was found that the eastern soils are better suited to rendering "available" the nitrogen of eastern sludges than are the California soils. Experiments were also made with California soils to determine the average "availability" of the nitrogen in several commercial organic fertilizers. The results are summarized in Table 160, and the general conclusions drawn by Lipman and Burgess are as follows:

"Chemical composition is of relatively little consequence as an indication of the fertilizing value of sludge nitrogen. If, as seems altogether possible, the sludge from various forms of septic and Imhoff tanks can be air-dried

and ground, it will have great value as a fertilizer if the nitrifiability of its nitrogen, as explained by us above, can be at all employed as a criterion. For California soils, especially, materials like sludge are of the greatest value since they will permit of the addition to our 'humus-poor' soils of plenty of cheap organic matter and nitrogen, and not only cheap but of a high degree of availability. These are two of the most important considerations in the soil fertility problems of this state.

"It is not merely the low nitrogen content of the sludge which brings out the marked contrast (in the percentage of available nitrogen) just discussed, since the absolute amounts of nitrate produced from sludge nitrogen are often 50 to 75 per cent. as high as those produced from similar weights of dried blood or high-grade tankage."

Table 160.—Partial Composition of Air-dry Sludges and their Available Nitrogen Compared with that of Commercial Organic Fertilizers

(Experiments by C. B. Lipman and P. S. Burgess, University of California)

Material	Volatile matter, per cent.	Ash, per cent.	Total N., per cent.	Nitrate N., per cent.	Phosphoric acid, per cent.	Percentage of N. in sludges and fertilizers available when inoculated with		
						Davis soil	Oakley soil	Anaheim soil
Sludge:								
Orange, Imhoff tank (city)	49.68	50.32	2.66	0.012	1.11	32.50	32.30	27.20
Fullerton Imhoff tank	25.31	74.69	1.23	0.045	0.86	43.90	43.50	40.60
Anaheim Imhoff tank	33.09	76.91	1.54	0.115	0.99	32.40	36.00	40.20
Lindsay septic tank	42.92	57.08	1.83	0.090	0.89	28.70	18.00	18.80
Pasadena Imhoff tank	29.34	70.76	1.68	0.135	1.46	38.00	28.20	35.70
Orange Imhoff tank (county)	38.41	61.59	2.38	0.060	0.77	25.70	21.40	15.70
Worcester exp. Imhoff tank	43.86	56.14	2.10	0.010	1.82	26.90	12.40	34.50
Cleveland exp. Imhoff tank	36.37	63.63	1.44	0.000	1.28	32.90	8.30	44.10
Chicago Stock Yards exp. Imhoff tank.	50.46	49.54	1.73	0.400	1.46	24.50	9.80	10.10
Fertilizers:								
Dried blood	......	......	......	......	......	12.79	0.00	4.05
High-grade tankage	......	......	......	......	......	16.21	0.00	3.95
Low-grade tankage	......	......	......	......	......	27.39	22.70	43.89
Fish guano	......	......	......	......	......	15.11	trace	4.65
Cottonseed meal	......	......	......	......	......	14.18	2.00	21.45
Goat manure	......	......	......	......	......	4.89	3.50	10.39

Sludge Fertilizers.—Many attempts have been made to produce commercial fertilizers from sludge and several are now being worked in Great Britain. The best known product of this sort is probably the "native guano" made from sludge from the chemical precipitation tanks at Kingston-on-Thames. It was widely advertised at one time at about

$25 a ton; Voelcker stated that samples sent to him showed it to be worth from $3.62 to $8.11 a ton. (*Journal Royal Agricultural Society*, vol. vi, 2d series, page 415.) The sewage is treated by the ABC process, alumino-ferric, blood, charcoal and clay being employed as precipitants. The sludge is pressed, partly dried, passed through sieves and then air-dried in storage. The composition of the resulting material is given in Table 161. A similar material is manufactured from sludge produced by using lime and ferric sulphate as precipitants at the Dalmarnock

TABLE 161.—PERCENTAGE COMPOSITION OF TWO FERTILIZERS MADE FROM SLUDGE

(Fifth Report, Royal Commission on Sewage Disposal, page 160)

	Native guano		Globe fertiliser
	A	B	
Moisture at about 110°C	25.87	10.19	22.51
Matter volatile on ignition	37.99	45.08	33.98
Non-volatile matter	36.14	44.73	43.51
Total	100.00	100.00	100.00
Matter insoluble in HCl after ignition	22.33	28.04	10.75
Oxides of iron and alumina.[1]	10.10	11.59	13.42
Lime[2]	3.30	4.78	12.09
Magnesia	....	0.42	....
Potash, soluble in dilute HCl	0.16	....	0.10
Potash, soluble in water	0.06	....	....
Phosphoric acid (P_2O_5)[3]	1.74	2.35	1.11
Equivalent to tribasic phosphate of lime	3.30	5.13	2.42
Yield of nitrogen, total[3]	1.93	2.49	1.30
Nitrogen evolved as ammonia on boiling for 2 hours with a 0.5 per cent. solution of potash.	0.41	0.50	0.06

[1] These oxides will contain the phosphoric acid and possibly a very little of the lime. [2] The lime was precipitated only once; it will contain traces of magnesia. [3] The figures for phosphoric acid and for total nitrogen are in each case the mean of two closely agreeing estimations.

works of the Glasgow sewerage system. This sludge is pressed, dried at a temperature of 65° to 70°C., and ground in a pan mill. It is known as "Globe fertilizer" and costs $2.40 a ton to make, exclusive of interest and sinking fund charges. Its composition is given in Table 161. This plant also sells its pressed sludge and its business success received the following comment in the Fifth Report of the Royal Commission on Sewage Disposal:

"It will be seen, therefore, that at Dalmarnock, by careful organization and advertising, practically the whole of the sludge is disposed of by sale, either as pressed cake or as Globe Fertilizer, the cost of the sewage treatment being in this way materially reduced. The saving effected is about 4s. per 1,000,000 gal. (Imperial). During the 3 years ending March 31, 1907, the whole of the pressed cake was sold to farmers. The demand is still increasing and the connection is now good enough to obviate the necessity for advertising. Mr. Melvin states that it is more convenient to sell sludge as pressed cake than as Globe fertilizer, partly because less work is entailed in the former case, and partly because in the manufacture of the fertilizer much dust is produced and he finds it difficult to get his men to wear respirators. For this reason, the Globe fertilizer is now made during spring and autumn only, and the quantity is diminishing year by year. In comparing the sale of pressed cake at the Dalmarnock works with the sales and possibilities of sales elsewhere, it has to be specially noted that there is a branch line of railway into the Dalmarnock works and, therefore, no carting is required" (page 162).

At Bradford, England, where the sewage contains a large quantity of grease from wool washing, amounting to about 440 parts per 1,000,000, the sewage is treated with sulphuric acid to precipitate the grease otherwise remaining in the effluent, and then settled. The sludge is treated with more acid, heated to about 100°C. and pressed in hot filter presses to extract the grease. Part of the pressed slud e is used for fuel, part is sold to farmers for about 84 cts. a ton at the works, and part is dried and disintegrated, selling for $1.32 to $1.60 a ton.

The treatment of the Travis tank sludge at Norwich was outlined on page 507.

The use of sludge for fuel, which is practised in other cities than Bradford, is of interest in connection with the attention now paid to a process of manufacturing fertilizer from sludge at the sewage outfall works of Dublin, conducted under the patents of the Anglo-Continental Fertilizers Syndicate, Ltd. The following notes regarding it are based on a description in John D. Watson's presidential address before the Institute of Sanitary Engineers in 1914. The sludge is screened and about 0.5 per cent. by weight of ordinary yeast added to it. This is warmed in a heater and delivered to 8 fermenting troughs, each about 50 ft. long and 4 ft. wide, holding about 3600 gal. These are kept at a temperature of about 90°F., and the sludge is kept in them about 24 hours. When the fermenting process is ended, the water is drained away at the bottom, leaving a sludge with a water content of 82 to 84 per cent. An amount of phosphates and potash about equal to the weight of the solid matter of the fermented sludge is then mixed with the latter in order to produce a good commercial product. This mixture is pumped into the top of a vertical dryer through which air at 450°F. is circulated.

The dried product is blown from the bottom of the heater by hot air into a pulverizing machine, which produces the powdered fertilizer.

A large amount of heat is required in this process, and it is proposed to supply this heat at other installations by burning sludge in destructors. The two processes combined have been estimated in one important project considered in 1914, to cost about 8 to 9 cts. per ton of wet sludge treated, apparently exclusive of capital charges and depreciation.

The production of fertilizer at Oldham, which has a population of 150,000, by the process developed by Dr. J. Grossmann, of Manchester, was begun in October, 1912, and proved so satisfactory that in 1914 the Local Government Board sanctioned an additional loan to complete parts of the plant which were not provided in the initial works. The process is one of drying the sludge and then passing superheated steam slightly acidulated through it to take up and remove the fatty acids. Grossmann states that the sewage is unusually poor in grease, containing only 8 per cent. calculated on the amount of dry sludge. If there were more, its sale would materially help pay the expenses of the process. The dry powder has found a ready sale, and the elimination of grease from it is stated to improve its fertilizing value materially. Three 3-men shifts working 8 hours each are required at the plant.

ENGLISH EXPERIENCE WITH SEWAGE FARMS

The sewage farms of England were planned very largely as a result of the practice of the Local Government Board to require some sort of land treatment of sewage and the effluents from other types of treatment works, as a condition upon which communities were authorized by the Board to raise money for sewage treatment undertakings. As the only other way to raise the money was the very costly one of obtaining a special act of Parliament sanctioning a loan, this alternative was rarely chosen. The Local Government based its practice on the reports of various commissions mentioned in Chapter I and was bound to insist on such a rule, according to the interim report of the Royal Commission on Sewage Disposal (page 8). The Royal Sewage Commission appointed in 1857 reported that "the right way to dispose of town sewage is to apply it continuously to land" (third report, March, 1865), and the committee appointed by the Local Government Board in 1875 reported "that town sewage can best and most cheaply be disposed of and purified by the process of land irrigation for agricultural purposes, where local conditions are favorable to its application." This committee pointed out, however, that the chemical value of sewage is greatly reduced to the farmer by the fact that it must be disposed of daily throughout the year, and its volume is generally greatest when it is of the least service to the land. The committee also stated that land irrigation was impracticable

in all cases and other modes of treatment must be allowed, which the Local Government Board apparently interpreted as meaning that other methods were not equivalent in sanitary value to land treatment, for it usually required the latter as a supplementary treatment in works approved by it.

The amount of land available for really successful sewage irrigation in England is quite limited, and as intensive methods of treatment were developed, strong opposition arose to the rulings of the Local Government Board. When the Royal Commission on Sewage Disposal was appointed in 1901, it early gave this subject detailed attention. A large number of sewage farms were visited and eight were selected as typical of different soil conditions. These were:

Soil	Farm	Soil	Farm
Sand..........	Aldershot camp	Heavy loam and clay	South Norwood
Gravelly loam.	Croydon (Beddington)	Heavy loam and clay	Leicester
Light loam....	Nottingham	Peaty soil and sand	Altrincham
Light loam....	Cambridge	Heavy loam	Rugby (high level farm)

These farms were kept under close observation for more than 2 years and observations were made less regularly at other farms. The detailed work was under the direction of Dr. G. McGowan, Dr. A. C. Houston and G. B. Kershaw and full accounts of it were published in four appendices to the fourth report of the commission. These furnish the most complete information regarding the actual operation of sewage farms which is now (1915) available. In using it elsewhere, however, careful attention should be paid to the effect of differences in soil and climatic conditions, for the generally moist equable climate of England, even though the actual rainfall is not high, makes the maximum quantity of water which land can take up very much less than the maximum quantity in places having a warmer and drier climate.

General information concerning these farms is given in Table 162. It will be observed that from 12 to 53 per cent. of the area was not irrigable, either because it was unsuited for the purpose or had not been prepared to receive sewage, or was required for roads, sites for buildings and sludge beds, grazing and such purposes. The ratio between the total irrigable acreage and the acreage required to treat satisfactorily the normal flow of sewage indicates the actual reserve capacity of the farm to permit the land to rest after receiving sewage, and to provide for

TABLE 162.—INFORMATION CONCERNING EIGHT ENGLISH SEWAGE FARMS, 1900 AND 1901
(From Reports of the Royal Commission on Sewage Disposal)

Place	Aldershot camp	Altrincham	Beddington (Croydon)	Cambridge	Leicester	Nottingham	Rugby (high level)	South Norwood
Date of construction	1864	1870	1861	1895	1891	1880	1867	1864
Total acreage	138	75	673	102	1699	907	40	191
Irrigable acreage	120	35	420	74	1350	651	35	152
Average acreage irrigated at one time	40	17	70	18	337	300	7	50
Population draining to farm	20,000	18,000	100,000	50,000	197,000	258,584	6000	21,000
Population per irrigable acre	166	514	238	675	146	397	171	138
Sewage per capita daily, U. S. gal.	60	53	48	54	44	32	60	34
Sewage per acre irrigated simultaneously, U. S. gal.	30,000	55,200	68,500	149,900	25,800	28,000	51,400	14,400
Sewage per irrigable acre, U. S. gal.	10,000	27,600	11,400	36,500	6,400	12,900	10,200	4800
Composition of sewage, parts per million:								
Total nitrogen	133.0	35.6	72.0	57.0	81.0	77.0	97.0	52.0
Total organic nitrogen	51.0	22.9	21.0	19.0	23.0	32.0	33.0	15.0
Albuminoid nitrogen	16.2	6.2	9.0	9.0	12.0	18.0	17.0	7.0
Oxygen absorbed[1]	208.0	52.5	125.0	101.0	224.0	232.0	184.0	77.0
Chlorine	150.0	94.3	83.0	85.0	134.0	137.0	100.0	75.0
Solids in suspension	366.0	....	345.0	265.0	341.0	520.0	473.0	219.0
Dilution of effluent	6	3	12	15	1	160	130	⅓
Composition of effluent, parts per million								
Total nitrogen	60	22.2	22.0	20.9	25.3	22.7	23.1	23.0
Total organic nitrogen	....	2.6	2.6	2.1	4.5	....	....	....
Albuminoid nitrogen	2.6	1.3	1.4	0.9	2.0	0.3	1.8	1.0
Oxygen absorbed	27.2	17.5	14.1	8.1	25.0	1.9	14.1	14.4
Bacteria per cc., averages:								
Gelatine at 20°C.	183,260	363,400	1,413,200	711,476	532,777	[2]	637,133	778,322
Reduction, per cent.	99	97	95	94	95	....	97	98
Agar at 37°C.	37,308	7,275	112,000	78,327	70,500	[2]	81,526	35,157
Reduction, per cent.	99	99	97	94	95	....	97	99
B. coli and coli-like microbes	1000–10,000	100–1,000	....	1000–10,000	1000–10,000	[2]	1000–10,000	100-1000
Spores of B. enter. spor.	10–100	1–10	10–100	10–100	10–100	[2]	10–100	10–100
Average annual rainfall, in.	22	37	24	21	20	25	26	24
Net annual cost of treatment, per 1,000,000 gal., including loan charges, based on dry-weather flow.	no loan charges $7.73	$4.70	$22.51	$9.17	$22.73	....	$5.99	$62.43 including pumping
Net annual cost of treatment per head of population draining to farm, including loan charges	$0.16	$0.08	$0.38	$0.18	$0.36	....	$0.12	$0.78

[1] Oxygen absorbed from permanganate in 4 hours at 80°F. [2] With few exceptions the effluents were remarkably good.

unusually heavy demands upon the farm. Theoretically it should also be proportional to the character of the land, a farm containing heavy soil requiring a larger reserve capacity than one with a light soil, but in practice this relation is difficult to observe, financial and topographical conditions obscuring its effect.

The average acreage irrigated at one time, taken in connection with the dry-weather quantity of sewage, gives the approximate volume of sewage actually treated per acre. Under English conditions the area is greater in the case of farms where surface irrigation[1] is practised than in the case of those using methods of distribution, with efficient under-drainage, called "filtration" in England. There is no agreement among sewage farm managers as to whether it is better to apply sewage heavily to a limited area and give long intervals of rest, or to apply it lightly to a larger area and give short periods for resting. There is also a great variation in English practice between the lengths of each working period and each resting period. This is important, because a ratio of 1 unit of working area to 2 units of irrigable areas, will be tabulated whether the period of working and resting is measured in days, weeks or months. Moreover, the momentary condition of the land and crop is likely to be of controlling importance.

The population draining to a sewage farm should be measured by the number per irrigable acre rather than by the number per acre of the entire farm. The difference by which the former exceeded the latter was often quite large, ranging from 21 in the case of Aldershot Camp to 274 in the case of Altrincham.

The amount of sewage per capita daily was decidedly variable in the case of the farms under consideration. The figures in the table were based on the dry-weather flow, and the provisions regarding storm water at the 8 farms were as follows: very little storm water is treated at Aldershot Camp, Altrincham, Cambridge and Nottingham; about two and one-half times the dry-weather flow is treated at Leicester; nearly all the storm water is treated at Rugby; and all the storm

[1] Considerable confusion has arisen regarding the meaning of some of the terms applied to sewage farming in England. The usage of the last Royal Commission has been to designate the flowing of sewage over inclined land, from an upper contour to a lower one, as surface irrigation. The sewage was not considered to penetrate more than a few inches into the soil in this method. Where the sewage was applied by any method to soil of such a nature that the sewage percolated downward through the material, the process was termed filtration. This is not in accord with earlier English usage. Ridge irrigation is practised by flooding sewage over land from channels on ridges between long flat slopes; the sewage collecting in the low places between the ridges is received by a cross ditch at the foot of the field and conducted to ridges in a field lying somewhat lower than the first, and where the soil is very heavy, a third field may be laid out. In bed irrigation or land filtration, as the term is used in France and Germany, the sewage is conveyed in numerous ditches to the land, which is not flooded but is kept moist by the sewage seeping sideways from the ditches. This is known as ridge-and-furrow irrigation in England. In flood irrigation, a plot of land surrounded by a bank is covered with sewage to a depth of 1 to 2 ft. In subsurface irrigation the sewage is distributed through open-joint pipe lines laid about a foot below the surface.

water which gains access to sewers is treated by the farms at Beddington and South Norwood.

The dose of sewage per acre in 24 hours, given in the table, was obtained by dividing the quantity of sewage in 24 hours in dry weather by the average area irrigated at one time. As a rule, the filtration farms treated much more sewage than those where surface irrigation was practised. It should also be pointed out that the dose of sewage per acre in 24 hours must be considered in connection with the ratio of irrigable acreage to average acreage irrigated at one time, for heavy dosing influences the time the land must rest and consequently the proportion of the irrigable land which may safely receive sewage at one time. It is sometimes desirable to have the general rate of sewage application, obtained by dividing the dry-weather flow per day by the total irrigable area, and these figures are accordingly given in Table 162. These figures do not represent the volumes of sewage per acre being treated daily, and are mainly instructive in showing the acreage required for sewage farming in comparison with other methods of treatment. A comparison of the rate per acre actually irrigated and the rate per acre of irrigable land gives one view of the ratio of working and resting time of the land, an important but complicated feature of the management of sewage farms.

The classes of sewage used at the farms were quite different, as is indicated in Table 162. That for Aldershot Camp and South Norwood was domestic exclusively; that for Rugby, Beddington, Cambridge and Altrincham was mainly domestic; that for Nottingham was four-sevenths domestic and three-sevenths trade refuse; and that for Leicester was three-fourths domestic and one-fourth trade refuse. The Aldershot Camp sewage was classed as very strong; it was screened and allowed about an hour's stay in settling tanks. The Altrincham sewage was classed as weak; it was given only a slight opportunity for sedimentation, as the tank had a capacity of but one-eightieth of the dry-weather flow, but the 21-in. outfall sewer was on such a flat grade, 1 : 2,174, that considerable silting occurred in it. The Croydon sewage delivered at the Beddington farm was screened and a part was given a brief period for sedimentation. The Cambridge sewage was classed as rather weak; it was screened and given half an hour for settling. The Leicester sewage was screened and pumped. A part of the Nottingham sewage was screened before being pumped, which was all the preparation any of it received. The Rugby sewage was treated with alumino-ferric, lime and sulphate of iron, and given a short period of sedimentation; part of it was screened; twice a day, a large amount of oil from machine shops was skimmed from the tanks, as rye-grass irrigated with sewage containing it suffered injury from its presence. The South Norwood sewage was screened through 1-in. and ¼-in. racks and given a little more than an hour's sedimentation.

Surface irrigation was employed mainly at Beddington, Leicester, Rugby and South Norwood, but some filtration was also practised. Filtration was employed at Aldershot Camp, Altrincham, Cambridge and Nottingham. The reason for the choice of method is evident when the prevailing soil and subsoil of the farms is considered, for "filtration," to use the English term, requires a lighter soil and better drainage than surface irrigation. The different farms possess soils of very different characteristics, and can be arranged in the following order, according to their value for treating sewage.

Filtering farms:
- Nottingham, sandy loam and gravel overlying sand and gravel.
- Cambridge, sandy loam overlying gravel and sand.
- Aldershot Camp, coarse sand overlying very fine sand.
- Altrincham, peaty soil overlying sand and gravel.

Irrigation farms, with some filtration:
- Beddington, gravelly loam overlying gravel and sand.
- Rugby, heavy loam overlying stiff clay with gravel pockets.
- Leicester, stiff clayey soil overlying dense clay.
- South Norwood, clay soil resting on London clay.

The importance of operating the farms so as to turn out good effluents varied as shown in the line in Table 162 giving the extent to which these effluents were diluted by the streams into which they were turned. The greatest dilution was in the River Trent, into which the effluent from the Nottingham farm was discharged, and the least, only 33 per cent., was in Chaffinch Beck, receiving the South Norwood effluent. As a matter of fact, the brooks or streams at about half the farms were not in a satisfactory condition, but in most of these cases the cause was pollution of the water above the farms. The bacteriological examinations showed that the effluents, except that from the Nottingham farm, usually contained from 100 to 1,000 B. coli or coli-like microbes, and 1 to 10 spores of B. enteritidis sporogenes per cubic centimeter. The percentages of reduction in bacteria by land treatment are given in the table.

The bacterial investigations also showed that the soils of sewage farms contained streptococci, B. coli and B. enteritidis sporogenes in great abundance; these organisms are relatively absent from virgin soil. The sewage bacteria do not crowd out the characteristic soil bacteria, it was stated, and the latter become prominent when the application of sewage to the land is discontinued. Repeated investigations had shown, according to this report, that the effluents from land processes of sewage treatment were not only characterized by the bacterial flora of sewage, but were relatively free from soil microbes.

McGowan, Houston and Kershaw reported that the land was overworked in most cases. The application of 30,000 U. S. gal. per acre per day at Aldershot Camp was too large a quantity of strong sewage to be treated by screening, settling and filtering through 3 ft. of coarse sand

overlying very fine sand, if uniformly satisfactory effluents were desired, although this rate would yield effluents showing a high percentage of purification. At Altrincham, the bacterial results were considered as showing that 55,200 gal. of weak settled sewage per acre of actually dosed land per day was not too great a rate for filtration through 3¾ ft. of porous peaty soil on sand with a little gravel; the chemical results were interpreted as showing the land to be somewhat overtaxed. The bacterial results of irrigation at Beddington with a screened sewage of medium strength on light gravelly loam over gravel and sand at a rate of 68,500 U. S. gal. per acre per day, were generally less satisfactory than the chemical results. The rate was reported to be too high and filtration was recommended in place of irrigation. The rate of 145,900 gal. at Cambridge was regarded as rather large for weak screened and settled sewage filtered through nearly 5 ft. of uncropped sandy loam on gravel and sand. At Leicester, where the rate was 25,800 gal., this was judged too high for irrigation with screened and settled sewage of moderate strength on stiff clayey soil over dense clay. The Nottingham farm was one of the few where the rate, 28,000 gal., was not considered too high; the sewage was classed as strong, and was screened and then filtered through about 6 ft. of sandy loam on sand and gravel. The Rugby farm was treating strong sewage on heavy loam over stiff clay, and the rate, 51,400 gal., was considered too high for irrigation with such sewage, even after it was screened and settled. The rate of 14,400 gal. at South Norwood was considered as possibly too large for irrigation with screened and settled sewage on lightened clay soil resting on London clay.

Commission's Conclusions.—In its fifth report, the Royal Commission on Sewage Disposal makes the following statements regarding the results of its investigations:

"There is no essential distinction between effluents from land and effluents from artificially constructed filters. Effluents from those soils which are particularly well adapted for the purification of sewage contain only a very small quantity of unoxidized organic matter, and are usually of a higher class than effluents from artificial filters, as at present constructed and used. Effluents from soils which are not well adapted for the purification of sewage may often be very impure" (page 231).

"Generally speaking, the evidence points to a maximum rate of 30,000 gal. (36,000 U. S. gal.) per acre, or 1000 persons per acre, with the best land, after preliminary treatment, although some witnesses put the rate as high as 60,000 gal. (72,000 U. S. gal.) per acre, or 2000 persons per acre, under similar conditions. With unsuitable land, such as clay, not more than 3000 gal. per acre can be efficiently treated, even after settlement of the sewage" (page 143).

"The total acreage of a farm must be relatively much greater when the sewage is purified by surface irrigation than when the method of filtration

is employed, and a larger percentage of surplus area is also desirable in the former case. We are not able to lay down any rule as to what the ratio of surplus acreage to total acreage should be, but generally speaking a large surplus area is advisable. It is true that the larger the total irrigable area, the greater is likely to be the working cost. On the other hand, with good management, the larger the surplus irrigable area, the better is the purification likely to be and—within certain limits—the prospect of profit" (page 145).

"In the case of sewage which is to be treated upon artificial filters, it is generally desirable to settle out as much of the suspended solids of the sewage as possible, before filtration. This is of less moment in the case of land, but where the soil is heavy we think that the sewage should usually be efficiently screened and settled. Porous sandy soils, worked as filtration farms, may be able to treat crude unsettled domestic sewage without detriment, but, even in those cases, there is the possibility of nuisance arising from the decomposition of sewage solids on the surface of the soil, and such solids may cause damage to crops" (page 149).

"It is impossible to lay too much stress on the importance of sewage farms being well managed. In this connection it has to be borne in mind that farm managers have a most difficult part to play, and no amount of care and attention will ever enable land, of any kind, to deal with a volume of sewage which is in excess of the effective purifying area of the soil. We think it would be useful that farm managers should be taught some simple test or tests to enable them to follow the operations of the land; that their instructions should include a definite order to consider the farming results as quite secondary to the production of an effluent of the required standard; that the statistics of the farm should be carefully kept; and that, wherever possible, the flow of sewage and storm water should be gaged throughout the year. As a general rule, we consider that sewage farms should not be let" (page 151).

"The soils and subsoils which we have considered may be divided into the following three broad classes: class I, all kinds of good soil and subsoil, *e.g.*, sandy loam overlying gravel and sand as at Nottingham, Cambridge and Beddington; class II, heavy soil overlying clay subsoil, as at Rugby; class III, stiff clayey soil overlying dense clay, as at Leicester and South Norwood. Since variations exist in practice, both as regards the method of purification employed and the extent of cropping, the first of these three classes may be divided into three sub-classes, as follows: sub-class (*a*), filtration with cropping; sub-class (*b*) filtration with little cropping; sub-class (*c*) surface irrigation with cropping. The methods of purification assumed for the other classes of soil are: class II, surface irrigation with cropping; class III, surface irrigation with cropping. With a heavy soil and clay subsoil, by far the greater part of the purification is effected by surface irrigation, though in exceptional circumstances, such as obtain at Leicester, a good deal is also done by filtration.

"Having regard to the volume of sewage dealt with and the purification effected at the farms which were kept under our own observation, and the evidence given by various witnesses, we estimate that the several classes

of soil and subsoil could effectively deal with the volumes of settled sewage given in Table 163."

TABLE 163.—AVERAGE AMOUNTS OF LAND REQUIRED FOR TREATING A DRY-WEATHER FLOW OF 1,000,000 U. S. GALLONS

(Royal Commission on Sewage Disposal. Fifth Report, page 153)

Class of soil	Method of working	Volume of settled sewage which can be treated per acre per 24 hours	Total area of land required to treat a dry-weather flow of 1,000,000 U.S. gal.[1]
I. Good soil and and subsoil	Filtration with cropping	14,400 U. S. gal.	70 acres
I. Good soil and subsoil	Filtration with little cropping	30,000 U. S. gal.	33 acres
I. Good soil and subsoil	Surface irrigation with cropping	8,400 U. S. gal.	121 acres
II. Heavy soil on clay	Surface irrigation with cropping	6,000 U. S. gal.	167 acres
III. Stiff clayey soil on dense clay	Surface irrigation with cropping	3,600 U. S. gal.	278 acres

[1] These areas are sufficient for the treatment in times of storm of three times the mean dry-weather flow.

Management.—The management of sewage farms in England is recognized as a special branch of agriculture, and the Royal Agricultural Society has occasionally made special studies of the subject. There has plainly been a tendency to subordinate the sanitary to the agricultural purposes of such farms, and this feature of sewage irrigation is one of its drawbacks even where it has some standing as an economic proposition.

In discussing the management of a sewage farm, stress is always laid by English authors on the importance of keeping down weeds. The experience of Lieut.-Col. Alfred S. Jones, who managed some of the most successful English farms, led him to recommend dairying as the best practical branch of agriculture for most cases. The reason for this was that rye-grass, mangels and other fodder crops are particularly suited for sewage-irrigated land and there is always a market for milk. The cultivation of the land for such crops may be regarded as little more expensive than keeping fallow land free from weeds. He kept his cows in stables which were maintained in a clean condition. If cattle are allowed to graze on sewage-irrigated land, they may do a great deal of damage to carriers and to ploughed channels, and for this reason, they are usually considered undesirable unless they can be kept away from the land which receives sewage.

In his "Sewage Disposal Works," Hugh P. Raikes terms weeds "a perpetual plague, which, if allowed to spread unchecked, will rapidly cover the whole ground, and by excluding light and air will prevent the purification of sewage discharged on it." He looks upon crops merely as a source of revenue to counterbalance a part of the cost of frequent working of the ground necessary to keep it well aerated and fit to receive sewage. He regards hand trenching or double digging with a fork as the most efficient form of cultivation; all strong-growing weeds should be shaken out and collected in piles for burning. Manual labor is usually too expensive, however, and most cultivation is done with horses, although on some large farms, like that belonging to Birmingham, steam cultivation has been practised.

When the land was worked to its full capacity, occasional signs of sewage "sickness" or foulness were inevitable, according to Raikes, even under the most skillful management. These occasional unsatisfactory periods were not considered serious by him. On the other hand, if the land received so much sewage as to become continuously sick, even a period of resting did not always restore it to a good condition and it was necessary to grow a grain crop, keeping all sewage off the tract.

To avoid pools of sewage dotted over a field, the grading of the surface and of the ditches and channels must be correctly carried out. Lieut.-Col. Jones has stated that most of the disfavor into which sewage farming fell was traceable to "the common absence of sufficient regularly contoured and neatly cut distribution carriers, and often to the manager's dependence on a borough surveyor's coming to the farm with his level and staff for great measures, or on his own guesses for smaller works, instead of using an instrument to peg out every distribution carrier at the right moment."

The main carriers are usually constructed of masonry or concrete, but for the minor distributing system earth channels are preferred, because they do not interfere with the cultivation of the land by horses, which is considered necessary even where no crops are grown, in order to promote aeration and check weeds. According to Raikes, filtration tracts in England are generally portions of irrigation areas temporarily utilized for this method of treatment for a year or two. Then another area is utilized in this way and the older one again employed for irrigation farming. This practice makes temporary carriers preferable to permanent ones except for main lines.

Much more care is paid to grading the land for filtration than for irrigation in England, although it is well recognized that even with irrigation, pooling of the sewage in detached places is likely to result in sewage sickness of the land at such points and in a poorer effluent. The cost of preparing the land for filtration is much greater than for surface irrigation, not only on account of the necessity for having the surface

perfectly level but also because it is frequently desirable to strip off the fine surface soil lying over a coarser subsoil. The preparation of such areas and their maintenance in a level condition free from weeds is one of the most important features of the work of the managers of some farms.

In laying out land for surface irrigation, Kershaw stated in his "Modern Methods of Sewage Purification," that the main point was to have a perfectly smooth surface for the sewage to flow over, the slope being just enough to cause the sewage to creep forward at a uniform rate. For a heavy soil and subsoil he advised a flatter slope than for more pervious land. All embankments should be made during the summer, in his opinion, because they will then have an opportunity to become consolidated; if constructed in the winter the frost is liable to throw them enough to allow sewage to escape through cracks and holes.

With surface irrigation the size of the plots or fields is not usually regarded as of much importance, because the sewage can be kept upon any portion of the land by regulating its distribution through the sluices of the carriers and by the grips or small channels formed on the surface. The average size at Leicester was 13 acres and at South Norwood 2 acres. While it is considered desirable to provide permanent carriers where it is certain that filtration will be practicable, it is customary to allow for considerable flexibility in such matters, because a competent manager will learn by experience where filtration can be conducted profitably and will construct the necessary carriers and embankments with a better appreciation of the local requirements than an engineer can have before the land has been operated as a sewage farm. Kershaw sums up his views as follows:

"In the case of surface irrigation farms it is generally necessary to provide subsidiary carriers over the surface of the land from the main carrier outlets; these are usually formed first of all with a ridge plow, and subsequently shoveled out and trimmed up by manual labor. Plow-formed grips for the sewage should not be continued to the end of the plot, but should run out just before the bottom of the plot is reached; otherwise, in the event of a storm, the sewage is washed over into the effluent channel at the foot of the plot.

"The method of distribution in use at the Beddington sewage farm is as follows: longitudinal channels or grips, about 50 ft. apart, are cut down the greatest fall of the land, the surface intervening between the grips being dished to cause the sewage to distribute over the surface more efficiently. These trenches run out about 50 yd. before the foot of the plot is reached, and the dishing stops, the surface of the ground thus being level at the foot of the plot. A pick-up carrier runs along the foot of each plot, into which the surface effluent passes.

"Generally speaking, surface irrigation requires more main carriers and subsidiary carriers than filtration, and, to distribute sewage efficiently on a

porous soil and subsoil, far more subsidiary carriers are needed than for a comparatively impervious soil, especially in the case of land which is laid out in large plots. The smaller the plots the easier the distribution of the sewage becomes. As a rule, where the land to be sewaged is hilly, contour grips are necessary, or small channels cut in the soil, following approximately the contour of the land, but diverted from the true contour line sufficiently to give a very slight fall." ("Modern Methods of Sewage Disposal," page 201.)

The underdrainage of land receiving sewage is regarded as having a dual object by many English engineers, the aeration of the soil and the removal of the sewage after it has percolated through the soil. The drainage is usually carried out by deep ditches into which lines of 3 or 4-in. porous tile drain discharge. It is held by some managers and engineers that the open ditches should be put in so as to keep the level of the ground water below the level of the tile drains, in order to keep the soil well aerated. Another practice often recommended in the case of very porous soil is to put in underdrains sparingly until the need for them is indicated. This recommendation is based on the unsatisfactory conditions usually left where drains are removed, due to the impracticability of restoring the soil to its natural conditions. The general opinion is that all land except clay needs underdrainage when the ground water lies within 4 ft. of the surface. The underdrainage of a number of English farms is given in Table 164, from Kershaw's "Modern Methods of Sewage Purification." Clay soil is difficult to underdrain because the tile seem to increase the number of cracks which open in dry, hot weather and permit raw sewage to enter the drains in an unchanged condition. Such soils are frequently lightened by plowing cinders or ashes into them, and if they are covered with turf the tendency to crack is minimized. Sand is not regarded favorably for sewage farming in England, because of the difficulty of securing a satisfactory effluent from it and at the same time raise crops of value. Where the sand contains iron, the sewage has sometimes formed a "pan" or impervious layer in the subsoil when the underdrainage was not properly carried out. The backfilling of the trenches in which tile have been laid must be done with particular care when sewage farming is practised, to avoid the danger of unchanged or little changed sewage reaching the tile.

There can be no fixed rules for applying sewage to farms operated on the so-called filtration system, according to English opinion. The manager must be permitted to learn by experience the best method of resting and working each field. The following general statements are given in Kershaw's "Modern Methods of Sewage Purification."

"Probably with porous soils and subsoils worked on filtration principles, the most satisfactory plan is to apply a heavy flush of sewage for from 6 to 12 hours, allowing a rest of from 12 to 18 hours.

"At the Nottingham sewage farm (filtration), excellent results are obtained by running the sewage continuously on to the land for 12 hours during the day, the remaining 12 hours constituting a resting period; the land is treated in this manner for about 1 month, or a little longer as the case may be, and then a fresh lot of land is brought into use.

"At the Leicester sewage farm (surface irrigation and filtration) in 1900 the settled sewage was applied to the land in a heavy flush the first day, and then in moderate volume continuously day and night for about 14 days; this plan was found to counteract the tendency of the soil to crack in hot weather.

TABLE 164.—UNDERDRAINAGE OF ENGLISH SEWAGE FARMS

(From G. Bertram Kershaw's "Modern Methods of Sewage Purification)

Place	Nature of soil and subsoil	Sewage per acre per 24 hours. U. S. gal.	Tributary drains			Main drains	
			Size, inches	Depth, feet	Spacing, feet	Size, inches	Type
Aldershot Camp	Sand	10,000	3	3¼, average[1]	36	4 to 15	Socketed
Altrincham...	Black, porous soil over sand and gravel....	27,600	4	3½ to 4	12 to 15	9	Socketed
Banbury.....	Light, gravelly loam overlying clay......	....	3 & 4	4	30 to 50	9	Socketed
Beddington (Croydon).	Gravelly loam over sand and gravel.......	11,400	4	4 to 9	Usually very wide apart	Up to 24	Socketed
Birmingham..	Variable......	....	3 & 4	4½ minimum	33 to 50	9 to 18	Socketed
Cambridge...	Sandy loam over gravel and sand.....	36,500	4	3¼ to 6	Chiefly 50[2]	12 to 24	Socketed
Crewl.......	Heavy red marl	....	4 & 6	6	20	....	....
Doncaster....	Light sandy loam with gravelly subsoil..........	....	....	3 to 6	66	....	....
Nottingham..	Light sandy loam and gravel overlying gravel and sand.........	12,900	4	4½ to 7½	66[3]	12 to 24 132 ft. apart[4]	Socketed
Rochdale....	Alluvial and gravel.......	....	4 & 6	5	30	....	....
Rugby.......	Heavy loam overlying clay	10,200	2	3¾	66	4 & 6	Butt
Stretford.....	Alluvial.......	....	4	3 to 5	60	6 to 12	....
Swanwick...	Clay soil......	....	4	4	15	......	....

[1] The available fall does not admit of a greater depth. [2] The spacing of the underdrains on two plots is only 14 and 16½ ft. [3] The spacing on a special filtration area is 33 ft. [4] The spacing of the main drains on the special filtration area is 66 ft.

"At the Cambridge sewage farm (filtration) the plots of land used as natural filter beds are sewaged continuously for about 6 hours a day; the remainder of the filtration area in use is flooded continuously for from 4 to 7 days, when the supply of sewage is cut off, and the land allowed to rest for as long as possible" (page 211).

EXPERIENCE IN GERMANY

Sewage irrigation in Germany has been on a very different footing from that in England. It has been looked upon more generally as a method of treatment, and no attempt has been made, except in cases too unimportant to be recorded in German technical literature, to irrigate heavy soil with sewage. There are about 50 cities having sewage farms. As a rule the sewage is distributed through the fields in furrows, separating the land into beds about 3 ft. wide and up to about 125 ft. long. Before sandy soil is laid out in this way it is sometimes flooded with sewage to a depth of 10 to 20 in. during the winter, as often as seems possible. This results in a deposit of sludge on the top of the soil, and this sludge is subsequently worked into the soil, with a view to improving its character for farming. At Magdeburg, the sewage is sprayed over grass meadows by hose connecting with portable 3-in. cast-iron pressure mains laid on the surface of the ground. In a few cases surface flooding by the ridge method is employed. The general preference seems to be to keep the sewage away from plants except as it can reach their roots by percolation through the soil. The amount of sewage treated per acre per day at several German sewage farms is given in Table 165, page 690, by Bredtschneider and Thumm (Mit. der Anstalt, Part IV).

Berlin.—The practicability of treating the sewage of Berlin on land lying in several directions from the city was one of the reasons leading to the adoption of the radial system of sewerage there. The farms went into operation about 1884 and were extended until their area at the close of 1910 was as follows:

	Farmed by city	Leased to farmers	Unproductive	Total
Area irrigated and farmed, acres	16,657	3,956	395	21,008
Area farmed without irrigation, acres	10,647	2,486	8,868	22,001
	27,304	6,442	9,263	43,009

The population at that time was 2,064,000 and the average amount of sewage was 77,000,000 gal. or 37 gal. per capita daily. Of the prepared land, 7994 acres were used for broad irrigation, 12,250 acres for filtra-

tion, 502 acres for settling basins, and 2105 acres for roads and miscellaneous purposes, making a total of 22,851 acres. The rate of filtration was about 3700 gal. per acre of prepared land per day. The principal crops were rye-grass, turnips, beets, cabbages, potatoes and grain. About a fourth of the area was operated by the city as pasturage, and there were about 40 acres of fish ponds which furnished fish at the rate of $80 per acre annually. The cost of the farms to March 31, 1910, was $17,470,000; subdivided as follows:

	Based upon	
	Total area	Land specially prepared
Purchase of land, per acre	$229.38	$431.52
Preparation of land, per acre	131.40	247.80
Buildings and miscellaneous, per acre	45.42	85.26
	$406.20	$764.58

This information is from the 1914 report of the Metropolitan Sewerage Commission of New York, which gives the expenses for the year ending March 31, 1910, as made up of $1,300,385.24 for maintenance and $741,817.62 for payment of interest and loans. The receipts were $1,240,772.58 and there was an estimated increase of $122,593.50 in the value of live stock and other property.

The land about Berlin is quite uniform in character and is used for sewage treatment by Charlottenburg, Steglitz, Rixdorf, Pankow, Reinickendorf and other cities with about 1,000,000 population. According to Allen Hazen (*Engineering News*, September 16, 1897) the soil is a light brown sand with an effective size of 0.13 mm. and a uniformity coefficient of 2.5. The top soil is thin and differs so little from the lower material that it is usually considered unnecessary to separate the two in grading operations. The thin growth of low pine which covers much of the land is cut off when a tract is prepared for farming, the ground is usually graded to a level surface with the help of light portable railways, and 2 to 3-in. underdrains are put in at depths of 4 to 6 ft. These drains are on grades as low as 1 : 250 in some places and are not more than 30 ft. apart. They discharge into main drains 4 to 7 in. in diameter, which have minimum grades of 1 : 500. When the effluent reaches a quantity beyond the capacity of a 7-in. tile drain, it is carried away in open ditches. These have 1 : 1½ side slopes protected with brush and longitudinal rows of small stakes until willows planted along the edges of the banks develop enough to hold them.

The sewage reaches the farms through force mains with which small standpipes are connected. These are merely a few lengths of pipe set up vertically to contain floats. Each float has a rod rising from it, to the top of which a large target is attached during the day and a lantern at night. The standpipe also has an overflow which discharges on to the beds in the vicinity, but is not expected to come into action under ordinary working conditions. The watermen are expected to watch the target or light and when it rises, to open enough gates on the pressure pipes connected with the force mains to keep the sewage from reaching the overflow. These gates discharge the sewage into small brush-lined basins with a screen of brush to intercept the large floating material. These basins give off more odor than other parts of the farm, but are considered as satisfactory as any substitute that has been proposed, besides being very cheap to construct and maintain. From these basins the sewage flows through a system of open ditches or carriers formed in the embankments around the beds into which the land is subdivided. The bottoms of the carriers are above the beds, so that the channels can be drained when not in use. A thick growth of grass soon covers them and holds their form satisfactorily. Wooden boxes through the sides of the carriers, and wooden gates for closing the boxes and the carriers,

TABLE 165.—INFORMATION REGARDING GERMAN SEWAGE FARMS

(Bredtschneider and Thumm)

City	Quantities per acre		Character of land used for irrigation
	Sewage, gallons daily	Persons connected with sewers	
Berlin	3,850–4,800	105	Loamy and sandy
Breslau	6,200	190	Sandy
Brunswick	3,630	115	Loamy and sandy
Charlottenburg	13,900	485	Sandy
Cottbus	19,000	283	Sandy
Danzig	9,700	285	Dune sand
Dortmund	4,800	107	Sandy
Freiburg i. B.	6,840	97	Sandy
Liegnitz	5,340	161	Sandy
Magdeburg	5,020	174	Sandy and gravelly
Münster i. W.	5,450	140	Sandy
Rixdorf	5,240	220	Loamy

Note.—In Charlottenburg the sewage passes through sedimentation tanks and at Cottbus through a vertical tank or tower; in other cases it receives no treatment before passing to the land.

enable the watermen to control the discharge of the sewage over the beds.

The cost of land at a Berlin farm being developed during Hazen's visit in 1896 was stated to be $194 per acre, and the cost of preparing the land was stated as $126 an acre. The cost of preparation was made up of $39 for grading and embankments, $39 for draining and $48 for distribution system for raw sewage.

Although sewage farming has strong advocates among German engineers, it also has critics who predict its gradual abandonment. Dr. Dunbar states in his "Principles of Sewage Treatment," page 266, the opinion of these critics in the following words:

"I am convinced that it would be cheaper for many towns to abandon irrigation and replace it by artificial biological processes. It appears fairly certain that this will be the course of affairs as soon as the growth of the towns exceeds a certain limit. I do not doubt, for example, that many of us will live to see the day when Berlin will sell its irrigation farms for building purposes and construct artificial biological works in their place."

His statement of the results of land treatment at Berlin is given in Table 166.

TABLE 166.—RESULTS OF LAND TREATMENT AT BERLIN IN 1908

(Dunbar; parts per 1,000,000)

	Sewage	Effluent	Reduction, per cent.
Total residue	978.4	987.0	....
Loss on ignition	285.2	124.0	56
Oxidizability, potassium permanganate	333.7	33.6	90
Chlorine	283.8	232.7	10
Ammonia and albuminoid ammonia	99.5	2.3	98
Nitric acid and nitrous acid	....	146.6	....

SEWAGE FARMING IN FRANCE

Sewage farming is practised by about 30 French cities, according to Imbeaux's "L'Assainessement des Villes." The most important example is afforded by Paris; Rheims, Poitiers and Montelimar are other cities in the list. In most cases the sewage is used for irrigating grass land and little attention is apparently paid to the quality of the effluent. The first farm to receive all the sewage of a French city was developed at Rheims by a private company, which applied the sewage at a rate of about 9000 gal. per acre daily to a "thin layer of earth overlying fissured limestone." (Bechmann, *Trans. Am. Soc. C. E.*, vol. liv, part E,

page 195.) The authors were informed by Le Couppey de la Forest, secretary of the Society for Public Health and Sanitary Engineering, that in 1908 the sewage of Rheims amounted to 11,800,000 gal. per day and the area under cultivation was 1472 acres. This gives a rate of a little over 8000 gal. per acre daily.

Paris.—In 1868 experiments with sewage irrigation were undertaken at Gennevilliers. After a little local opposition, irrigation with Paris sewage at that suburb steadily increased in extent and was so satisfactory from both sanitary and agricultural standpoints that in 1889 permission was obtained to carry on sewage irrigation at Achères, and the farms were put in service in 1895. The legal average rate at which sewage can be applied to the land is 11,800 gal. per acre daily, but apparently this is sometimes considerably exceeded. In 1894, additions to the existing works were authorized, which were completed in 1899, and since then supplementary works were undertaken to enable most of the sewage of the city and part of its suburbs to be applied to land, except when the storm overflows are in operation to relieve the intercepters of part of the run-off from heavy rainfalls.

According to the 1914 report of the Metropolitan Sewerage Commission of New York, the population contributing sewage to the farms in 1910 was 2,800,000. The volume of sewage was then 160,000,000 to 185,000,000 gal. per day, equivalent to 57.1 to 64.8 gal. per capita. About 160,000,000 gal. were used for irrigation. The extent of the land receiving sewage at that time was as follows:

	Area of farms, acres		
	Privately owned	Owned by city	Total
Gennevilliers	1,996	15	2,011
Achères	386	2,965	3,351
Méry-Pierrelaye	3,731	1,235	4,966
Carrières-Triel	2,137	210	2,347
	8,250	4,425	12,675

The cost of the farms owned by the city was $7,220,000 in 1900 and the annual operating expenses of the farms and the distribution of sewage was about $1,000,000. The Gennevilliers crops in 1907 were reported to be worth about $400,000. Financial records of the farms are not obtainable.

According to Bechmann, the irrigated lands, except those of Méry, are alluvial deposits of very porous sand and gravel 6 to 20 ft. deep. Near the Seine the sand contains silt which makes careful underdrainage

necessary to keep down the ground-water level. The plateau of Méry is partly fine and medium sand and partly loam on limestone, making underdrainage necessary. The city's land is leased to tenants at rates ranging from about $5 per acre, where the tenant agrees to apply the sewage to land only as directed by the city's representatives, to about $40 per acre where he uses the sewage when and how he desires. The owners of the land take the sewage as they desire it. At Gennevilliers, where market gardening is mainly practised, the city maintains a model farm, to assist those using the sewage to apply it in the most effective ways.

The sewage is screened and settled. It is distributed through the farms in reinforced concrete conduits 1 to 4 ft. in diameter, lined with sheet steel where the pressures are heavy. These conduits have risers 1 ft. in diameter with outlets for the sewage into the open carriers which distribute it over the fields. The underdrains have an average depth of nearly 10 ft. and are constructed of plain or reinforced-concrete pipe. They discharge into open ditches with concrete lining.

The average results of land treatment at the Paris sewage farms in 1901 are given in Table 167, from Bechmann's paper previously mentioned.

TABLE 167.—RESULTS OF LAND TREATMENT AT SEWAGE FARMS OF PARIS IN 1901

(Bechmann; parts per 1,000,000)

	Raw sewage	Effluents from farms at			
		Gennevilliers	Achères	Méry	Carrières
Mineral matter	152.0	307.0	259.0	182.0	263.0
Organic matter	33.4	1.1	1.7	0.9	1.1
Chlorine	52.0	83.0	72.0	27.0	68.0
Nitrogen as nitrates	0.8	28.2	19.1	19.7	21.0
Ammonia	15.4	0.0	0.2	0.0	0.0
Organic	6.7	0.0	0.0	0.0	0.0
Bacteria per cc.	148,322,000	585.0	1,000	125.0	250.0

SEWAGE IRRIGATION IN AMERICA

Sewage irrigation in America had its beginning in small undertakings at the State Insane Asylum at Augusta, Me., and the Worcester, Mass., State Hospital. The latter attempt to irrigate with sewage attracted much attention for several years. The sewage first entered a covered storage tank 30 ft. long, 16 ft. wide and about 6 ft. deep. The inlet pipe had its end bent over and submerged a few inches, and the outlet

pipe was similarly bent over but carried down in the tank so that its end was about 2½ ft. below the surface of the sewage. There was a sludge drain leading to a pit where it was proposed to compost the sludge with other refuse, but there are no records to show this was ever done. About 34 acres were available for irrigation, which was carried on with satisfaction to the asylum authorities for a number of years, until the number of patients considerably exceeded the 600 for which the system was laid out. There is no record of the date when irrigation was abandoned. The superintendent informed the authors in 1914 that he understood sewage farming was given up on account of objectionable odors from the surface of the fields.

At the State Farm at Howard, R. I., where an irrigation system was constructed in 1885 from the plans of Samuel M. Gray, the history has been different from that of most of these early sewage farms, for the fields laid out then have been in continuous use to the present time. One field has an area of about 2.5 acres and the other an area of 5.7 acres. Both had originally about 10 in. of fine light loam on a subsoil of sand and fine gravel, growing coarser as the depth increased. The surface was leveled and lines of 3 and 4-in. tile drains were laid about 40 ft. apart at a depth of 5 to 6 ft. The sewage was screened and carried in channel pipe bedded in concrete on the top of embankments running around the sides of the land. This channel pipe had gates at intervals of about 100 ft., through which the sewage was discharged on the sloping fields. Dr. F. B. Jewett, superintendent, informed the authors in 1914 that it was the custom to turn the day sewage on the large lot and the night sewage on the other. Very little effluent appears at the outlets of the drains. Up to 1912, the land was cropped, vegetables being raised in the earlier years and corn subsequently. The sewage was distributed in furrows. Since 1912, no crops have been raised and intermittent filtration has been practised.

One of the boldest attempts at irrigation with sewage in this country was made by Col. Geo. E. Waring at Wayne, Pa., in 1891. It is of interest as an attempt to follow English practice where the land is not adapted for such work. The original site was described by Col. Waring in the *American Architect*, July 2, 1892, as follows:

"It consisted mainly of an old pond surrounded by ancient pollard willows, a large area of swamp through which the brook meandered, about 4 acres of slightly sloping cleared land, and a very steep, thickly wooded and rocky hillside, rising about 100 ft. from the level of the brook to one corner of the nearly square tract. The pond was obliterated, the willows and much other vegetation were cleared away, the brook was confined within stone walls, and all except the steep hillside was thoroughly underdrained."

The tract comprised 11 acres and was intersected diagonally by the brook, which was straightened and deepened, and its banks above the

stone revetment were sodded. The sewage applied to the land on one side of the brook was screened, and that for the remainder of the tract was both settled and screened. The sewage was discharged in each case behind a low bank of broken stone which was placed there to cause it to be distributed uniformly over the rather sharply sloping hillside. For the same purpose several low banks of locomotive cinders were laid on contours at successive intervals down the slopes. The sewage was somewhat checked behind these banks, and started uniformly at each. It could flow along the ground behind a bank more freely than through the interstices between the cinders and it was thus distributed across the face of the hill at each bank.

It was stated by M. N. Baker in *Engineering News*, Nov. 3, 1892, that during a visit to the place a short time before, the sewage was well distributed and caused no perceptible pollution of the brook, in which he saw small fish. Five crops of grass had been raised on each side of the brook and there had been scarcely any trouble with frost during the winter of 1891–92.

After the system had been in operation a few years it passed into the hands of the Springfield Water Co., and is now the property of the Wayne Sewerage Co., of which J. W. Ledoux is chief engineer. He informed the authors that broad irrigation did not prove successful, and in 1906 the company was forced to make radical changes on account of the many complaints of nuisance by residents near the plant. The changes were made under the direction of G. Everett Hill, and included the construction of a septic tank, preliminary coke strainer and intermittent sand filters.

The first American town of any size to try sewage irrigation was Pullman, Ill., now a part of Chicago. In 1880 George M. Pullman entrusted to the late Benezette Williams the planning of the sewers for this town, located on Lake Calumet, a body of water about 3 miles long, 1½ miles wide and not over 10 ft. deep at any place. The land was level and only 7 to 8 ft. above the lake, making it necessary to pump at least a part of the sewage. A separate system discharging into a 300,000-gal. covered reservoir was built, and the sewage pumped through a 20-in. force main nearly 3 miles long to a screening tank at the farm. The tank was a closed 6-ft. vertical cylinder 24 ft. high, elevated sufficiently to enable the screenings to be dropped into wagons driven below it. From this tank a main distributing pipe ran through the farm. It was provided with an overflow pipe and a pressure-reducing valve to prevent any pressure exceeding 10 lb. coming on the piping system through the farm, which was of vitrified pipe. The branches from the main were 9 in. in diameter and 315 ft. apart and were provided with hydrants every 320 ft., from which sewage was distributed over the ground by means of hose and temporary ditches. The land was underdrained by lines of 2

to 4-in. tile laid at a depth of about 3½ ft. and about 40 ft. apart. These discharged into a 6 to 12-in. main drain which emptied into a ditch running to the lake. The land had about 1 ft. of black alluvium on clay subsoil. It was utilized both by filtration, for which purpose about 15 acres were subdivided by embankments, and by irrigation on tracts used mainly for market gardening. During the latter part of the operation of the farm very little attention was paid to the quality of the effluent and it was stated by several engineers who visited the farm that they had observed raw sewage discharged into the lake. The history of the farm is recorded in *Engineering News*, vol. ix, page 203; vol. xxix, page 27.

Reports furnished to the authors by the health departments of many of the Eastern states record several cities and a number of small communities as practising irrigation with sewage but investigation shows that usually the sewage is subjected to intermittent filtration and the raising of crops is merely incidental. Growing corn on intermittent filters is a practice difficult for a thrifty commissioner to resist, but experience at South Framingham, Marlboro, Brockton, and other Massachusetts cities proves that, if continued for a number of years, it is likely to be destructive of the filters. The roots add to the organic matter deposited in the pores of the sand by the sewage and eventually the top part of the sand is changed to sandy loam, of little use for filtration. Beds of sand suitable for intermittent filters and properly situated for such work are generally far too valuable to be damaged by raising crops in addition to purifying sewage.

One of the few Eastern cities reported by the health bureaus as using sewage for irrigation, which actually did so in 1914, is Danbury, Conn., having a population of about 25,000. City Engineer A. L. Davis stated to the authors in 1914 that the city owned about 200 acres of land, of which about 10 acres were divided into beds each of one-third of an acre, used as intermittent filters. Most of the treatment was done on these. About 20 acres of fields were used for the irrigation. The filter beds were originally a low swampy muck hole, which was underdrained by 4-in. tile 20 to 25 ft. apart and then covered with 3 to 4 ft. of sand. Analyses of the effluent had been made monthly and no unfavorable reports had been received, according to Mr. Davis. The irrigation fields, which were about 200 ft. long and 30 to 50 ft. wide, were located on the slopes leading down to the filters. The sewage was carried around the crest of the slopes in 24-in. channel pipes, and as it flowed down the slopes it was intercepted by low ridges of broken stone, as at the irrigation tracts at Wayne. The Danbury works cost about $30,000 and Mr. Davis estimated the annual maintenance expenses at $3000 to $3500. The engineers were Waring, Chapman & Farquhar.

Irrigation is practised in connection with filtration at Fostoria, Ohio, a city of about 10,000 population with about 620,000 gal. of sewage

daily. There are 6 acres of filters and a 14-acre irrigation field. City Engineer Chas. Latshaw informed the authors in 1914 that the top soil of the field was 4 in. of loam under which was a mixture of 40 per cent. of clay, 30 per cent. of gravel and 30 per cent. of sand. This is underdrained by 5-in. tile 3 ft. deep and 20 ft. apart. The field is flooded from an 8-in. outlet discharging upon a pile of boulders at one of the embankments. About 40 bushels of corn or 1½ tons of hay per acre are raised on the field. The results of the treatment have not been satisfactory to the State Board of Health.

A sewage disposal plant which attracted considerable attention at one time was that at Hastings, Neb. The authors have received the following official summary of the history of that plant down to 1915, which is typical of the reports received concerning a number of irrigation enterprises formerly rather prominent in the pages of technical journals.

"About 20 years ago beds were prepared to receive crude sewage and some unsuccessful attempts were made to raise a few vegetables by irrigation, which has been abandoned. Since then some additional beds have been graded and put into use, and a small septic tank built. There has never been any skilful management of the plant. It has not yet become a sufficient nuisance to compel an effort at improvement to be made."

The last sentence gives the reason for the eventual abandonment of sewage irrigation in many places in the Western states, where it was at one time used to the practical exclusion of other methods of treatment. In 1893, *Engineering News* stated that the only Western cities with treatment works of any other type than irrigation fields, were Leadville, Colo., and Hastings, Neb., where crude filtration methods were then employed. One of the first Western cities to practice irrigation with sewage was apparently Cheyenne, Wyo. This was carried on from about 1883 to about 1890, when a change in the position of the sewage outlet led to its abandonment. At Los Angeles sewage was probably used for irrigating market gardens about the same time, or even earlier. It was unsatisfactory for this purpose for several reasons, and the practice was gradually abandoned. The leading objections were the offensive odors from the sewage-irrigated lands near suburban property, and the opinion which gradually gained ground that sewage did not give such good results as water in irrigation work. Salt Lake City is another place frequently mentioned by advocates of sewage farming as treating its sewage in that manner. The facts in this case were communicated to the authors in 1914 by City Engineer Sylvester Q. Cannon, as follows:

"The city leases all the sewage to a farmer, who makes very little use of it. The period of his lease is for 25 years from Nov. 30, 1903; the consideration, $1. The sewage not used flows through an open outlet ditch to Great Salt Lake 12 miles from the city."

In 1914, sewage irrigation was apparently practised to a greater extent in California than in any other state, about 35 places employing this method of treatment. Little official information concerning these farms is available but reports from engineers not connected with them indicate that irrigation is not conducted in an efficient manner as a rule, from either a sanitary or agricultural viewpoint.

Sewage irrigation at Pasadena, Cal., began about 1887, when 300 acres were bought at $125 per acre for a municipal farm. In 1905, 160 acres more were purchased at $150, and later purchases brought the total up to 518 acres in 1914. The property is 4½ miles from the city, from which the sewage is carried in a 16 to 20-in. vitrified pipe line. For some years the sewage was passed through settling basins, where the deposits were mainly rags, corks and coffee grounds, and was then delivered through pipes to outlets 400 to 500 ft. apart. These delivered the sewage into head ditches, from which furrows were run 2 to 6 ft. apart. A septic tank was constructed in 1910 and concrete pipes laid to distribute the sewage about the farm. The standpipes were placed about 150 ft. apart. The sewage was not allowed to run continuously on any area of open ground longer than 4 to 10 days, and as soon as the land was dry it was thoroughly cultivated and occasionally plowed. In the walnut groves the sewage was formerly kept on the land from Dec. 1 to April 1, while the leaves were off the trees. During the remainder of the year, the sewage was used exclusively on the open fields. The crops raised have been barley, wheat, hay, pumpkins, alfalfa, and English walnuts. Alfalfa has been profitable but the plants collected the solids in the sewage and its cultivation was accordingly abandoned for a time.

The condition of the farm in 1914 is outlined in the following notes from City Engineer Lewis E. Smith: Alfalfa was raised on 122½ acres; walnuts on 112 acres; oranges on 70 acres; oat hay on 120 acres; corn on 40 acres; pumpkins on 4 acres; sweet corn on 3 acres; kaffa corn on 2 acres. About 1,800,000 gal. of sewage were received daily, and after the septic tank was put in service there was no trouble caused by solids. Alfalfa is now one of the best crops and a cutting is made about once a month, the land being irrigated after each cutting. The walnuts are now irrigated about twice during the summer and twice during the winter.

Sewage irrigation at Fresno, Cal., has been carried on since about 1900. The authors have been unable to obtain any official information concerning it and the following notes are from the two journals mentioned hereafter. At first the city paid $5000 a year for permission to discharge its sewage over private land. In 1907, a municipally owned system was put in service, consisting of a 150 × 200-ft. septic tank divided into 8 compartments, each 36 × 90 × 8 ft. deep, and an 812-acre farm. The tank was designed to afford 8 hours' storage for

the maximum amount of sewage for which the works were intended to provide. The effluent was distributed by a main ditch from which furrows were ploughed. According to *Engineering and Contracting*, August 16, 1911, an application of 5000 to 15,000 gal. per acre per day was about all that most crops would stand. The most successful crops were carrots, parsnips, turnips, potatoes, peas, beans, oats, barley, wheat, corn, pumpkins, tomatoes, cabbages, English walnuts, alfalfa and Italian rye-grass. In January, 1908, 794 acres were leased for 10 years, with a 10-year renewal privilege, according to *Engineering Record*, August 22, 1908. The irrigable portion was to be planted to a vineyard, and the non-irrigable portion to fig, pomelo and orange trees. The city agreed to prepare the ground and construct all ditches. The lessees agreed to take entire charge of the effluent from the septic tank and distribute it over the land, subject to the control of the city. The total rental for the first year was $8000 and for each subsequent year $9000, more than enough to meet the interest on the bonds issued to pay for the disposal works.

At Pomona, Cal., the sewage is taken through an outfall sewer 6.59 miles long to a 100-acre municipal farm and adjacent rented land also irrigated with sewage. The sewage must be pumped to some of the main distributing ditches. The population is about 11,000 of which half is connected with the 20 miles of sewers in the city. City Engineer Clarence E. Bayley informed the authors in 1914 that the farm was just about self-sustaining.

The sewage of San Antonio, Tex., is conveyed 12 miles from the city to a 6700-acre privately owned tract, where it was used in irrigating 1500 acres in 1911, when the farms were visited by one of the authors. Ultimately about 4000 acres will be irrigated, it is expected. The sewage amounted to about 12,000,000 gal. daily, so that the land was dosed at the rate of 8000 gal. per acre daily. The irrigating season is from the middle of February to the middle of November and during the remaining 3 months of the year the sewage is stored in a lake having an area of about 1000 acres.

CHAPTER XVIII

AUTOMATIC APPARATUS FOR DOSING

It is expensive and, in many cases, impracticable to employ attendants to regulate the flow of sewage at treatment plants, especially where it is desirable to apply the sewage in doses. A number of automatic devices have been used to attain the desired regulation, with varying degrees of success. The principal types may be grouped, according to the treatment for which they are used, as follows: (*a*) for intermittent sand filters, (*b*) for contact beds, and (*c*) for trickling filters. In general the dosing apparatus acts as a valve to control the discharge of a dosing tank.

DOSING APPARATUS FOR INTERMITTENT SAND FILTERS

The apparatus under this classification may be divided into the following groups: (*a*) air-controlled siphonic apparatus, (*b*) mechanically controlled siphonic apparatus, and (*c*) mechanical devices.

Air-controlled Siphonic Apparatus.—The main feature of all siphonic apparatus is the siphon. Its principal parts, Fig. 185, are the main trap, a pipe casting with the long leg extending above the bottom of the dosing tank and the short leg connected to the discharge pipe to the filter; the bell, a cylindrical casting set over the long leg of the main trap and supported on legs or piers above the tank floor; the vent pipe, and the blow-off trap, made up of small galvanized wrought-iron pipe.

The main trap, immediately after the siphon has ceased discharging, stands full of water to the level B_1. The blow-off trap is also full to the level D_1. The vent pipe is empty. Sewage flows into the dosing tank and the water level rises until the open end of the vent pipe is reached at A. The vent pipe then becomes full of water and the siphon is sealed against the escape of air confined in the bell and upper part of the long leg of the main trap. As the water in the dosing tank continues to rise, it exerts a pressure upon the air confined in the siphon and forces the water in the long leg of the main trap down toward the lower bend. The water in that portion of the blow-off trap under the bell is likewise forced down. At the same time the water level inside the bell rises. Just before the discharge level in the dosing tank is reached, the water level in the blow-off trap is at D_2, in the main trap at B_2, and in the bell at C. A slight further rise of the water in the dosing tank forces the

seal in the blow-off trap, thus releasing the air confined in the bell and causing a sudden inrush of water from the dosing tank into the bell, which sets the siphon into full operation. The sewage in the dosing tank is discharged through the siphon until the level is at the low-water line at the lower bend of the vent pipe, when air is drawn into the bell through the vent pipe, the siphonic action is broken, the bell is filled with air, the discharge ceases, and the main trap and blow-off trap are refilled with water. The dosing tank then fills again and the siphon is ready for another discharge.

The air vent in the discharge pipe line, although not necessary for the working of the siphon, allows the escape of air previously confined in the bell and prevents trouble from air in the pipe line. There are a number of modifications of this form of apparatus, but the principles

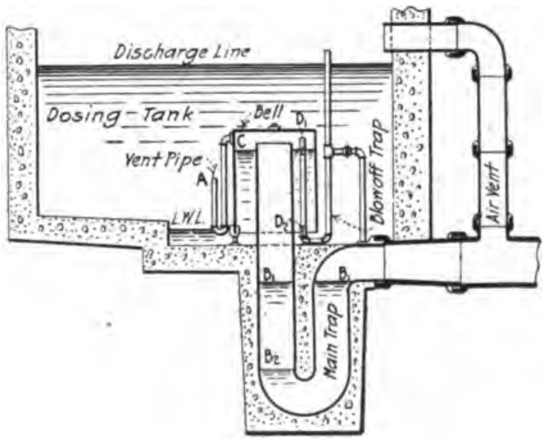


FIG. 185.—Siphon used in dosing intermittent filters (Miller).

are, in general, the same. The small siphons, 3 to 8 in. in diameter, in some cases do not require blow-off traps to ensure their working.

Where it is desirable to dose two or more filter beds in rotation, this can be done by installing several siphons, each connected to a filter bed and arranged to discharge in rotation automatically. Two siphons of the general type illustrated by Fig. 185, set side by side in a dosing tank, will operate alternately without special piping. For three or more alternating siphons, a special system of piping with starting bells or other controlling devices, is required. An installation of this character is shown in Fig. 186.

When this apparatus is first installed the main traps and blow-off traps are filled with water, also the starting wells of siphons 2, 3, and 4, leaving starting well 1 empty. The three-way air cocks are turned so that

air from each starting bell will be transmitted in the direction shown by the arrows.

As sewage flows into the tank and the water rises, it finally overflows into starting well 1, which is soon filled with water. This confines the air in starting bell 1 and the air pressure is transmitted, as shown by the arrows, to the blow-off trap of siphon 2. When the discharge line in the dosing tank is reached, sufficient pressure is thus exerted to force out the water seal in blow-off trap 2, which releases the air confined in siphon bell 2 and causes this siphon to come into full operation.

While siphon 2 is operating, siphonic action is developed in the drain-

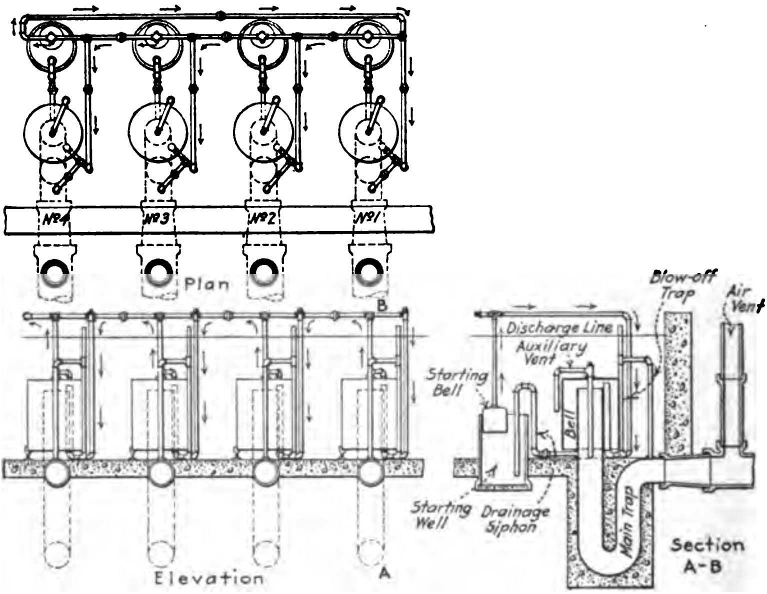


FIG. 186.—Four plural alternating siphons (Miller).

ing siphon connected with starting well 2, and as soon as the level in the tank is below the top of the well it is drained down to a point below the bottom of starting bell 2. After the first discharge, starting well 2 is empty, whereas the other three are full, well 1 having been filled when the tank was emptied by siphon 2.

When the tank is filled the second time, pressure is developed in starting bell 2 which forces the seal of blow-off trap 3 thus starting siphon 3; siphon 3 drains starting well 3, and on the third filling of the tank, starting bell 3 brings siphon 4 into operation. This siphon drains well 4 and on the fourth filling of the tank pressure is transmitted back to blow-off trap 1, bringing siphon 1 into operation. Thus the cycle of operation

continues. Any one, two or three siphons may be cut out and the remainder will continue to operate as before.

Another type of control for plural alternating siphons has been developed more recently in which no piping connecting the respective siphons is required. This is shown in Fig. 187. Each siphon is set separately from the others and is complete in itself, a fact which may be of advantage in irregular layouts.

At the start, the main and blow-off traps are full of water, the cut-out valves are closed and the regulating valves are open. The main siphon

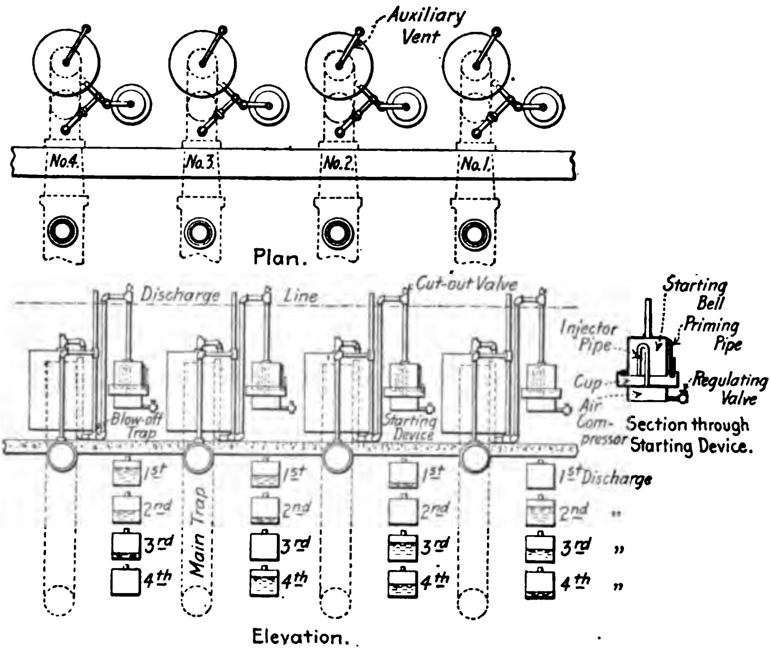


FIG. 187.—Four plural alternating siphons without connecting pipes (Miller).

is brought into operation by forcing the seal of the blow-off trap by pressure transmitted from a starting bell, and the head necessary to develop this pressure is the distance from the bottom of the starting bell to the discharge line, which in turn is equal to the depth of the blow-off trap seal. In other words, that siphon whose starting bell is completely filled with air will be brought into operation in advance of one whose starting bell is partially filled with water.

It will be noticed that the starting bell of siphon 1 has no priming pipe. The open end of the priming pipe on No. 2 extends one-fourth way up from the bottom of the starting bell, No. 3 priming pipe one-half

way up and No. 4 three-fourths up on the starting bell of that siphon. Therefore, when the sewage rises for the first filling of the tank, the starting bells will be sealed at the above-named points, that is, No. 1 at the bottom, No. 2 at the quarter point, No. 3 at the half point and No. 4 at the three-quarter point. When the discharge line is reached, starting bell 1 is full of air and the head developed forces the seal of the blow-off trap and brings the main siphon into operation. When the blow-off trap is forced, the air within the starting bell is released and the bell filled with water. Therefore, after the first discharge, bell 1 is full of water; No. 2 is one-quarter full; No. 3 is one-half full; and No. 4 is three-quarters full, and as the bottoms of the bells are below the upper edge of the cups (which are left full of water) these levels are maintained when the tank is empty.

In each of the air compressors is a regulating valve, and when all four siphons are in operation, these valves are open. The capacity of the air compressor above the regulating valve is equal to one-quarter the capacity of the starting bell.

As the water rises in the tank for the second time, the air from each compressor is forced through the injector pipe into its starting bell, thereby displacing one-quarter of the water. Bell 2 is one-quarter filled, which, on being forced out, leaves the bell full of air, and when the discharge line is reached for the second time, siphon 2 will be the one to operate. Bell 3 is now one-quarter full; No. 4 is one-half full and No. 1 is three-quarters full.

On the third filling, siphon 3 will have its last quarter displaced and will be brought into operation while the volume in the remaining bells will be reduced one-quarter. On the fourth filling siphon 4 will operate, and the fifth will return to No. 1, this cycle being repeated continuously. Fig. 187 shows the water levels in all four starting bells at each discharge.

If it is desired to cut out any siphon, this can be done by opening the cut-out valve of that siphon and closing the regulating valves in the air compressors of the remaining three. The opening of the cut-out valve prevents compression in that starting bell by opening it to the air. Closing the regulating valve increases the capacity of the air compressor so that it equals one-third of the capacity of the starting bell. Thus one-third of the water is displaced when three siphons are operating, instead of the one-quarter when four siphons are in operation.

Selection of Size of Siphon.—Siphons for dosing intermittent sand filters should have sufficient discharging capacity at the minimum head to empty the dosing tank, even under conditions of maximum rates of inflow from the trunk sewer. If such maximum rates are higher than it is feasible to provide for, the balance above a predetermined rate may be by-passed or the siphon may be allowed to go into continuous operation.

Operating continuously, the siphon will discharge at a maximum rate dependent upon the maximum head available. Table 168, prepared by the Pacific Flush Tank Co., is a guide in the selection of a siphon to cover this contingency. Fig. 188 indicates the dimensions referred to as "minimum" and "maximum heads." The minimum head can usually be varied only a slight amount for one size of siphon, but the maximum head can be varied to suit the conditions of each case. Table 169, also

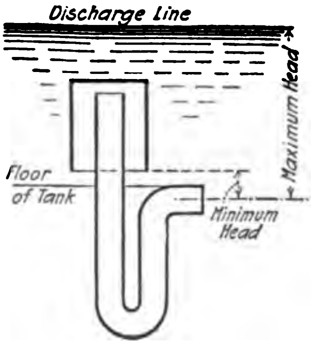


FIG. 188.—Position of maximum and minimum heads.

prepared by the Pacific Flush Tank Co., gives the rates of discharge for siphons discharging freely, that is, into an open trough or similar unrestricted channel.

TABLE 168.—SIZES OF SIPHONS TO BE USED WITH VARIOUS RATES OF INFLOW

Max. inflow in gallons per 24 hours	Diameter of siphon, inches	Minimum head, inches	Discharge, gal. per min. at minimum head
Up to 100,000	5	6	160
100,000 to 135,000	6	6½	230
135,000 to 280,000	8	7½	450
280,000 to 480,000	10	9½	800
480,000 to 720,000	12	10½	1,200
720,000 to 1,070,000	14	13	1,780
1,070,000 to 1,465,000	16	14	2,440
1,465,000 to 1,925,000	18	15	3,210
1,925,000 to 2,540,000	20	17	4,230
2,540,000 to 3,860,000	24	19	6,430
3,860,000 to 6,360,000	30	22	10,600

Table 169.—Rates of Discharge in Gallons per Minute for Siphons Discharging into an Open Trough

Head	Diameter of siphon										
	5 in.	6 in.	8 in.	10 in	12 in.	14 in.	16 in.	18 in.	20 in.	24 in.	30 in.
1'3"	255	345	640	1,000	1,420	1,950	2,560	3,210	3,960	5,720	8,950
1'6"	280	375	690	1,060	1,545	2,120	2,770	3,485	4,320	6,210	9,720
1'9"	300	410	750	1,180	1,675	2,300	3,010	3,780	4,680	6,740	10,530
2'0"	320	435	800	1,260	1,785	2,450	3,210	4,030	5,000	7,200	11,250
2'3"	340	460	850	1,340	1,900	2,605	3,410	4,290	5,320	7,650	11,970
2'6"	360	490	900	1,420	2,000	2,760	3,610	4,530	5,580	8,060	12,650
2'9"	380	510	945	1,490	2,100	2,890	3,780	4,750	5,900	8,450	13,230
3'0"	395	535	985	1,550	2,200	3,010	3,950	4,960	6,120	8,850	13,830
3'3"	410	555	1,020	1,600	2,275	3,130	4,100	5,150	6,400	9,200	14,350
3'6"	425	575	1,060	1,675	2,370	3,250	4,260	5,350	6,620	9,550	14,930
3'9"	440	590	1,100	1,730	2,455	3,370	4,410	5,540	6,880	9,900	15,470
4'0"	455	620	1,140	1,790	2,540	3,470	4,550	5,720	7,060	10,210	15,960
4'3"	470	635	1,170	1,840	2,610	3,580	4,690	5,900	7,300	10,520	16,440
4'6"	485	650	1,200	1,880	2,670	3,690	4,820	6,070	7,510	10,800	16,900
4'9"	500	670	1,235	1,950	3,760	3,790	4,960	6,240	7,700	11,120	17,380
5'0"	510	690	1,270	2,000	2,840	3,890	5,100	6,410	7,920	11,430	17,870
5'3"	520	710	1,300	2,050	2,900	3,980	5,220	6,550	8,100	11,700	18,270
5'6"	535	725	1,335	2,100	2,980	4,080	5,340	6,720	8,320	11,980	18,740
5'9"	545	740	1,365	2,145	3,045	4,170	5,460	6,860	8,500	12,250	19,120
6'0"	555	755	1,395	2,190	3,100	4,270	5,570	7,000	8,650	12,500	19,530
6'3"	570	770	1,420	2,230	3,165	4,340	5,680	7,150	8,820	12,750	19,930
6'6"	580	780	1,450	2,280	3,230	4,430	5,800	7,300	9,000	13,000	20,340
6'9"	590	800	1,475	2,325	3,290	4,520	5,910	7,440	9,190	13,260	20,720
7'0"	600	815	1,500	2,365	3,340	4,600	6,020	7,570	9,360	13,500	21,130
7'3"	610	830	1,525	2,395	3,400	4,680	6,130	7,690	9,500	13,700	21,430
7'6"	620	845	1,555	2,440	3,470	4,760	6,230	7,830	9,680	13,970	21,850
7'9"	630	860	1,580	2,480	3,520	4,840	6,330	7,930	9,820	14,190	22,200
8'0"	640	875	1,610	2,525	3,590	4,920	6,430	8,070	10,000	14,450	22,580
8'3"	650	885	1,630	2,560	3,640	5,000	6,530	8,200	10,180	14,640	22,890
8'6"	660	900	1,660	2,600	3,700	5,080	6,640	8,310	10,350	14,900	23,290
8'9"	670	910	1,680	2,640	3,740	5,160	6,730	8,430	10,500	15,100	23,600
9'0"	680	920	1,700	2,675	3,790	5,240	6,820	8,550	10,630	15,300	23,930

E. G. Bradbury gives[1] the following formula as furnishing a close estimate of the discharge of siphons under varying heads (Report Ohio Engineering Society, 1910, page 79):

$$Q = 0.4A\sqrt{2gh}$$

[1] A paper in *Cornell Civil Engineer*, June, 1912, by Weston Gavett, offers suggestions as to the design and selection of siphons as well as their maintenance, based on the results of experiments and also correspondence with engineers as to actual experience.

where Q = discharge in cubic feet per second.
A = area of discharge pipe in square feet.
h = average head in feet or half the vertical height from high water in tank to center of outlet.

Maintenance of Siphons.—Considerable trouble has been experienced in the operation of siphonic apparatus, a large part of which has been overcome by improved designs and greater simplicity of parts. In regard to the failure of siphons to operate properly, Bradbury states:

"When siphons fail to operate as intended, as is often the case, it is invariably because of some carelessness or oversight in setting, leakage of air, or stoppage of vents. In such cases, examine the outlet end to see that the water has an opportunity to fall clear of the end of blow-off tap; see that bell or intake end is at least 3 in. clear of the floor; clean out sniff hole or vent pipe; remove bell and see that there are no rags hanging on top of vertical limb in such manner as to draw water over by capillary attraction; if air piping is used and the siphon cannot otherwise be made to work, remove it, examine for cracks, and replace, tightening all joints well with white or red lead; and if there is still trouble, send for the manufacturer."

Barbour's Siphon Controlling Apparatus.—There are a number of devices to control the operation of siphonic apparatus in order that the discharge may be diverted from one pipe line to another automatically. One of the best known was devised by F. A. Barbour and installed at the sewage treatment works of North Attleboro, Mass., as well as a number of other plants. It is illustrated in Figs. 189 and 190.

The main features are an air-lock siphon controlled by an air valve and float, and a revolving cylindrical gate valve, also actuated by a float, the whole apparatus being set in the dosing tank. As the level of the sewage in the dosing tank rises, it lifts the large float A to which is attached a rack and pinion, causing the cylindrical gate valve to turn until the opening in the gate is opposite one of the outlet pipes to a filter bed or group of filter beds. At the same time, float B rises until the high-water level in the tank is reached, when the trip E on the float rod opens the air valve D, suddenly reducing the air pressure in the siphon bell and causing the siphon to come into full discharge. In case the trip failed to open the air valve, an additional rise in the water level would bring the siphon into full discharge by the aid of the pilot pipe, in a manner similar to that previously described. The siphon continues to discharge until the low-water level in the tank is reached, when air is drawn in under the edge of the siphon bell, the siphonic action is broken, and the discharge ceases. When float A falls with the sewage level, the cylindrical gate valve is not turned back, but is held in place by a pawl and ratchet on the pinion. The tank then fills again, raising float A, which turns the cylindrical gate valve around to the next outlet pipe, and the process

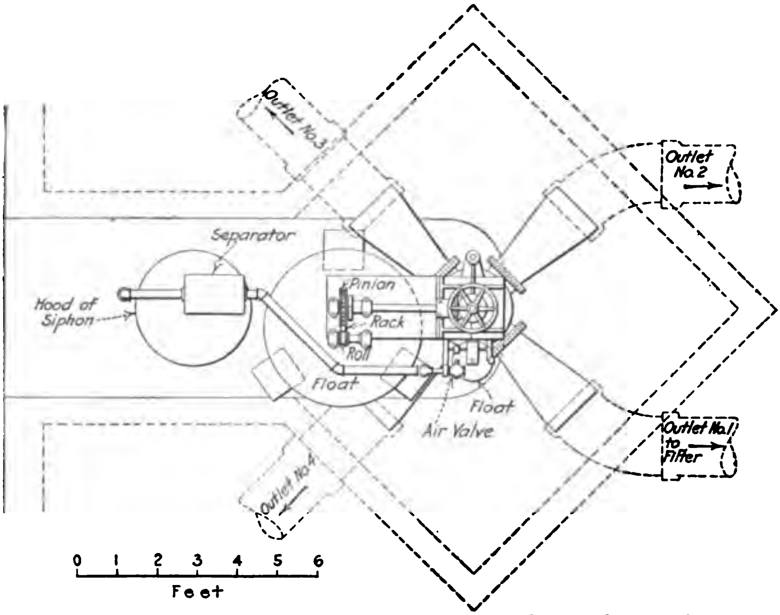


FIG. 189.—Plan of dosing apparatus at North Attleboro, Mass.

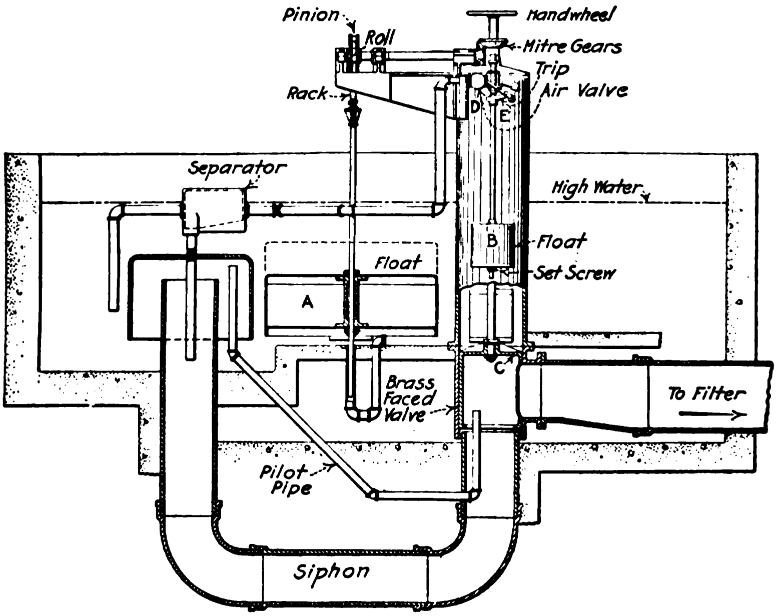


FIG. 190.—Sectional elevation of Barbour dosing apparatus at North Attleboro, Mass.

is repeated as described. The advantage of the small float *B* and air valve is that the size of the dose can be regulated by setting the float *B* higher or lower on the float rod. The action is positive and productive of more satisfactory results than when the pilot pipe is relied upon to start the siphon. The "separator" is necessary to prevent suspended matter from getting into the air valve. This apparatus is patented and the installation illustrated was made by The Atlantic Works, Boston, Mass., at a cost of $1525, which included erection in the building on foundations furnished by the town.

Shield's Siphon Controlling Device.—Another apparatus working on an entirely different principle is the rolling ball and siphon device designed by W. S. Shields and installed by Alvord & Shields at Lake Forest, Ill., and Wauwatosa, Wis. It is illustrated in Fig. 191.

The Lake Forest installation consists of ten 10-in. Miller siphons, set in a single dosing tank. Each siphon has a vertical box above it upon the floor over the tank and these boxes are connected with outgoing and incoming troughs so inclined that a metal ball will readily roll by gravity from the outlet of one box into the next one. It is necessary to elevate the ball a few inches in each box in order that the circuit may be continuous. Beneath each box is a float chamber containing a copper float with a small platform or elevator attached to the upper end of the float rod, as shown. On top of this elevator is a seat or cage into which the ball rolls from the incoming trough, lodging in such a position that when the elevator is raised by the float to a predetermined height, it will roll off into the outgoing trough.

In the bottom of each trough is a trip lever, so arranged that when the ball strikes it in its descent, it carries the lever down and opens the air valve connected with the air bell of a siphon, thereby releasing the air pressure and starting the siphon into operation. The ball remains on the end of the lever, resting against a guide on the side of the elevator, which is now at its maximum height. As the water is drawn from the tank the float descends until the cage is opposite the ball, at which point it rolls onto the elevator, releasing the trip lever and closing the air cock. The ball then goes down with the elevator to the bottom of the box where it remains until the dosing tank is empty and the discharge of the siphon ceases. The tank then begins to fill and the float and ball again rise until the ball rolls on to open the next siphon in the circuit.

At Wauwatosa, where six 10-in. siphons are operated, a 3-in. brass ball has been successfully operating the device from 1901 to date (1915) with but two interruptions. One was caused by a defective trough and the other by the freezing of the air valves, which were too tight. This latter difficulty was overcome by leaving the air valves quite loose. The device proved its efficiency particularly during the winter of 1914–15, when in a week of extreme cold the temperature dropped to $-20°F$. A revo-

lution counter operated by a copper float registers each discharge of the tank and consequently the quantity of sewage treated. The cost of the controlling chamber and device at Wauwatosa was $1326.

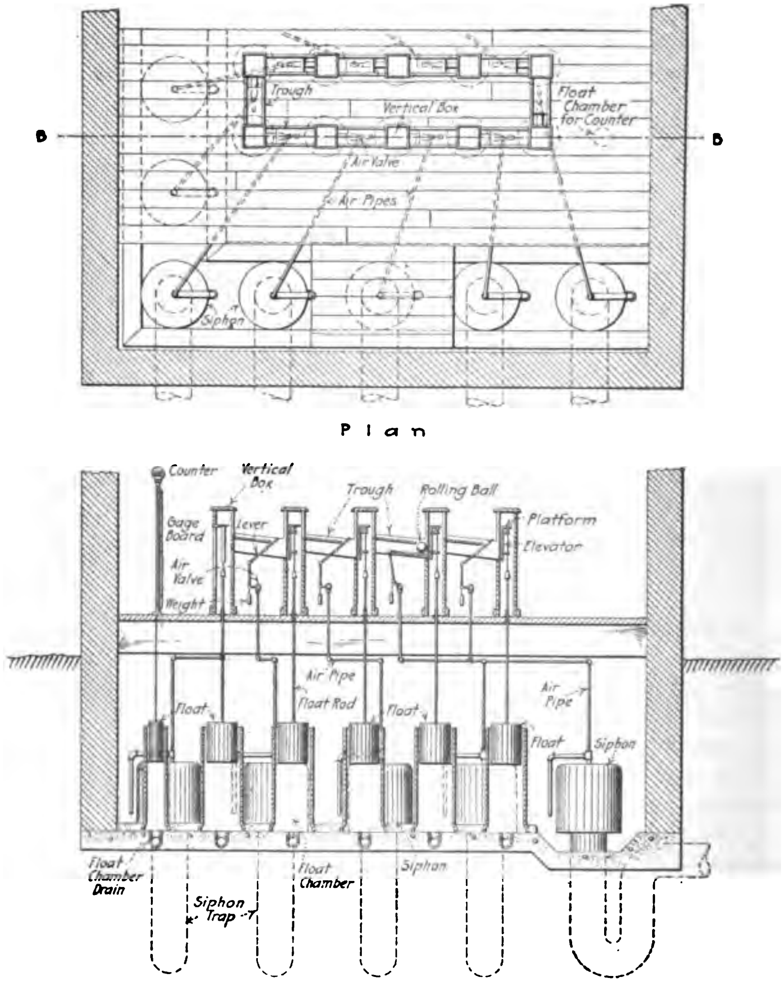


Fig. 191.—Shield's ball-controlled dosing apparatus.

Some rolling ball devices of this type are covered by letters patent Nos. 686913, 703090 and 793963 granted to W. S. Shields. These rights have been purchased by the Pacific Flush Tank Co.

Bradbury's Siphon Controlling Device.—In 1907 Bradbury designed an automatic sewage distributing apparatus which has been used suc-

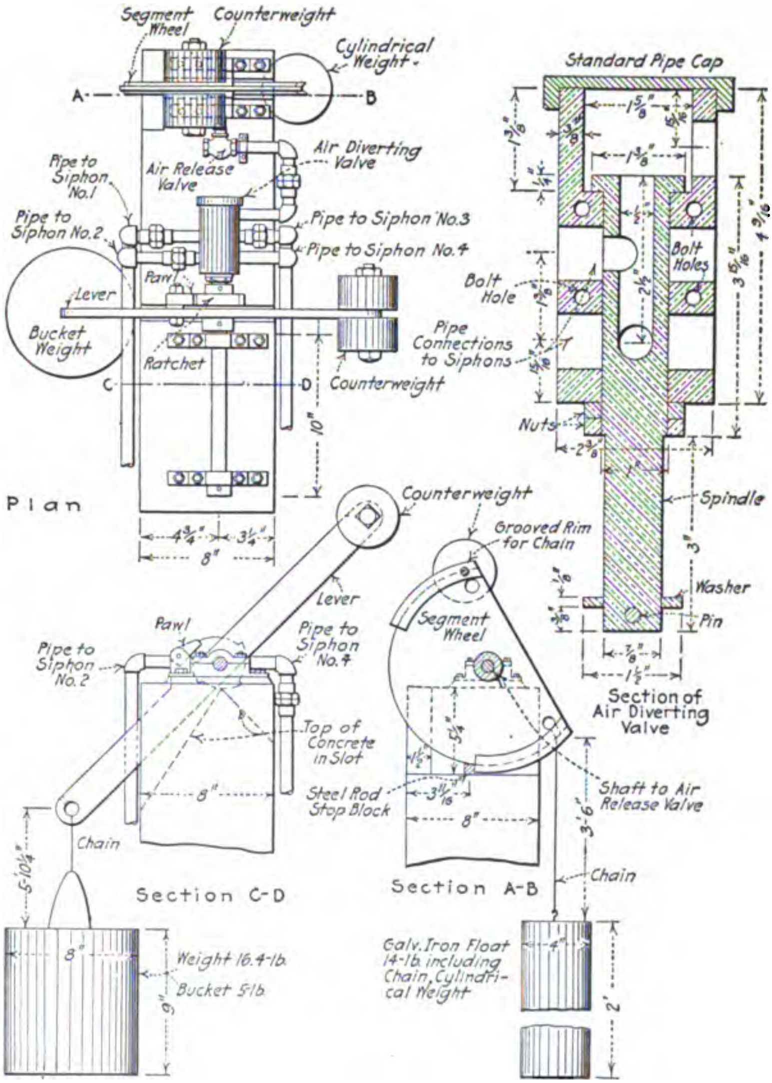


FIG. 192.—Bradbury's controlling apparatus for siphons.

cessfully at the Girls' Industrial Home at Delaware, near Columbus, Ohio, for mechanically releasing the air from a series of 4 siphons, thereby causing them to discharge in sequence. The complete appara-

tus consists of four 8-in. siphons of such dimensions that they will not discharge automatically under the head available; a four-way air-diverting valve operated by a float and lever; and an air-releasing valve controlled by a float and wheel arrangement.

The location of the air-diverting valve and its details are shown in Fig. 192. It consists of a square block of brass, drilled longitudinally, with 2 openings on each side connected by piping to the air bells of the 4 siphons, and 1 opening connected to the air-releasing valve. A brass spindle is fitted to the longitudinal bore so as to turn but remain air-tight. This spindle is drilled longitudinally and has 2 openings at right angles to each other so that they register with the 2 pairs of pipes leading to the siphons. The open end of the shaft communicates with the air-releasing valve.

This arrangement opens the 4 ports to the siphons in sequence by successive quarter turns of the spindle, which is operated by the bucket-weight and counterweight shown in Section C-D, Fig. 192. The bucket is kept full of water, being submerged at each filling of the tank. The movement of the float and spindle is gradual during the filling of the dosing tank, being, however, complete before the lowest discharging level is reached. The air-releasing valve remains closed during this period.

The air-releasing valve is a simple gas cock, opened by a quarter turn. The cylindrical weight operating this valve is made long and slender in order to get the proper relation between weight and displacement. It is not a float at all, being heavier than water. It is hung by a chain from the grooved rim of the segment wheel, as shown in Section A-B. The weight and wheel remain stationary until the tank fills to the proper height, which is near the top of the cylindrical weight. At this point the reduced weight of the cylinder is overcome by the counterweight and the wheel quickly makes a quarter turn, because the leverage of the counterweight increases more rapidly than does the weight of the cylinder as it rises out of the water. The air valve is thus opened and the air pressure in one of the siphon bells released, causing the contents of the tank to be discharged. When the water level has fallen about 2 ft., the weight of the cylinder overbalances the counterweight, and the operation is reversed, thereby closing the valve in the same manner, the relation of weights and leverage again causing rapid action.

At each filling of the tank the air-diverting valve makes a quarter turn, opening a different siphon, and the operation of the air-releasing valve is repeated.

The discharge height can be varied by changing the length of the chain by which the cylinder is suspended.

Mechanical Dosing Apparatus at Newton, N. J.—The sewage treatment plants at Newton, N. J., and Ravenna, Ohio, designed by Williams,

Proctor & Potts, are provided with automatic dosing apparatus of the type shown in Fig. 193, made by the Ansonia Manufacturing Co. Sewage flows through the settling tank into a dosing chamber containing a float. A chain attached to the float passes over a sprocket wheel, on the shaft of which is a weighted lever. As the sewage in the dosing tank rises, the float rises and the shaft revolves, bringing the weighted

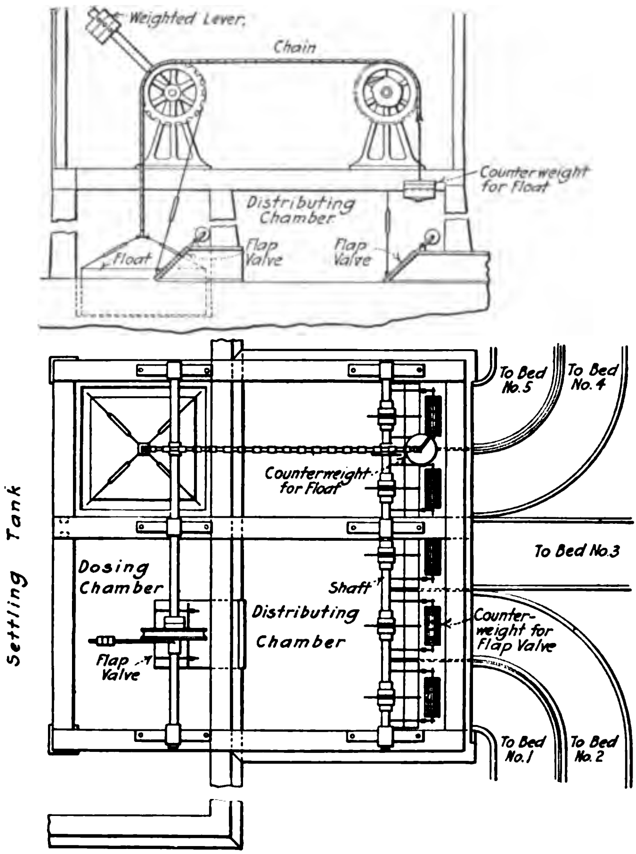


FIG. 193.—Dosing apparatus at Newton, N. J. (Ansonia).

lever to a vertical position. As soon as the shaft turns so as to bring the center of gravity past the vertical, the weighted lever falls on the opposite side and the flap valve between the dosing chamber and the distributing chamber is suddenly opened. Each rise of the float revolves the countershaft to which the distributing gates are attached, a fifth of a revolution, the fall of the float failing to revolve the shaft in

the opposite direction on account of the pawl and ratchet. The 5 flap valves leading to the 5 filter beds are each attached to the shaft at points equally spaced around the circumference, and thus at each one-fifth turn of the shaft a new gate is opened, the other four remaining closed. In this way the doses are distributed to each filter bed in succession.

This apparatus is readily adjustable and in general has been found to work satisfactorily. Care should be exercised in designing apparatus of this type to provide parts of sufficient strength to withstand the shock due to the suddenly applied force of the falling weight and the resulting jerk on the flap valves.

Float-controlled Centrifugal Pump.—Where it is necessary to pump the sewage from the tanks to the filter beds, the dosing can be automatically regulated by the control of the pump. This type of control was used by the authors as follows: The effluent from settling tanks flows over a weir into a dosing tank. As the water level in the dosing tank rises, it lifts a float in a float chamber of a pump well. When a predetermined elevation is reached a ball on the float chain lifts the lever of an automatic float switch, suddenly closes an electric circuit and starts a motor driving the centrifugal pump. The pump discharges the contents of the dosing tank onto the filter bed. The float then drops until the low-water line is again reached, when the float switch is opened and the pump stops, allowing the dosing tank to fill once more. In a similar manner the pump could be used to fill a dosing tank the discharge of which is controlled by a siphon. There are many variations of this form of controlling apparatus which are useful where sewage pumping is required.

DOSING APPARATUS FOR CONTACT BEDS

For the automatic control of contact beds, 2 sets of apparatus are required; one for filling the bed with sewage, and the other for emptying or discharging the sewage after a definite period of contact. For filling the beds, alternating siphons in dosing tanks may be used if sufficient fall is available. In this case the apparatus is similar to that already described. The dosing tank should have a capacity equal to the voids in the contact bed. Where there is only a limited head available, air-lock feed apparatus has been found satisfactory. Various mechanical devices have also been developed, working in conjunction with siphonic apparatus to fill and empty contact beds. For discharging sewage from the bed after contact, timed siphons have been extensively used.

The automatic apparatus for dosing contact beds may be grouped as follows: (*a*) air-lock siphonic apparatus, (*b*) mechanically controlled

siphons, (*c*) float-operated valves, and (*d*) miscellaneous mechanical devices.

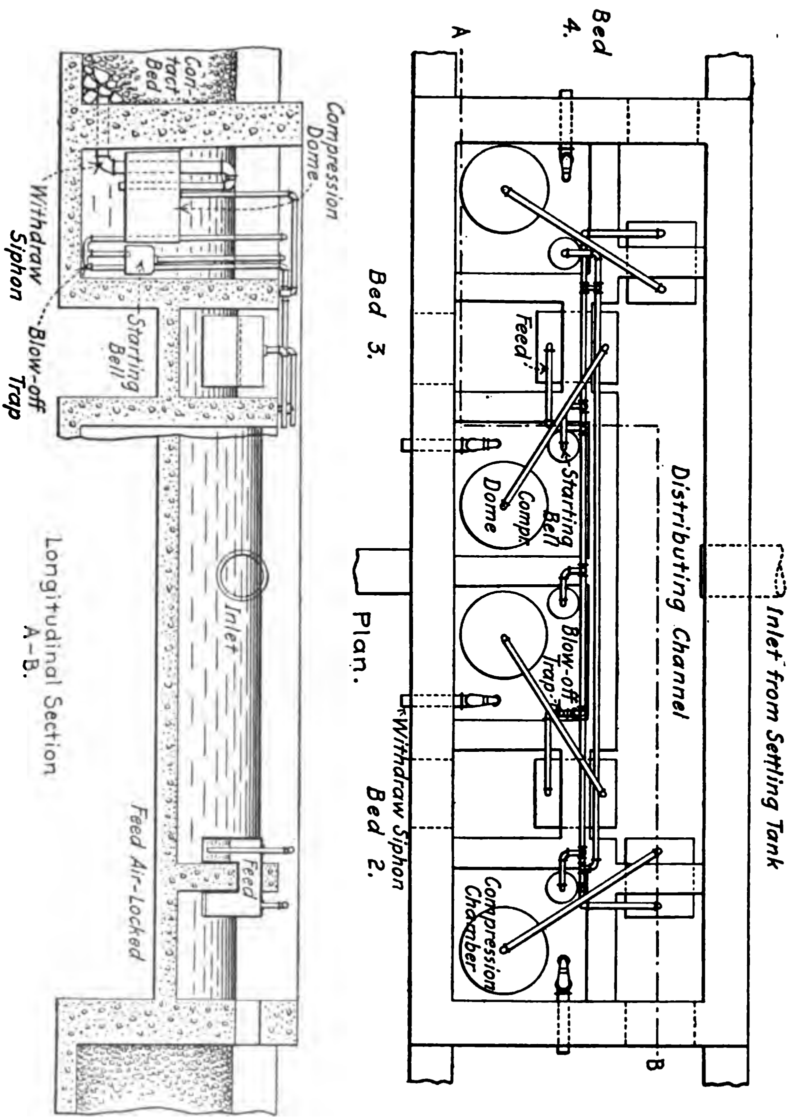


FIG. 194.—Air-lock feeds for contact beds (Miller-Adams).

Air-lock Feeds.—This type of apparatus operates in a different manner from the alternating siphons. Fig. 194 shows in outline the general features of an installation of air-lock feeds for 4 contact beds.

At the beginning of a cycle of operation the wells in front of the air-lock feeds are filled with water, except that which is to operate first. All of the blow-off traps are filled with water.

Sewage entering the channel from the settling tanks flows through the feed (the one not sealed with water) into the bed until it is filled, or until the sewage level reaches the top of the withdraw siphon. A slight additional rise in water level causes the withdraw siphon to come into operation, and the compression chamber is filled through the withdraw siphon until the sewage level inside is the same as in the bed. As the compression chamber fills, the air in the compression dome is put under pressure and forced into the upper part of the feed, gradually displacing the sewage flowing through the feed until the sewage level is forced down below the inside crest of the feed, when the flow through the feed ceases and the feed is air-locked.

The same rise of sewage in the compression chamber also produces a pressure in the starting bell, which is transmitted to the blow-off trap of the feed next to operate. Just before feed 1 is air-locked, the seal in blow-off trap 2 is forced, thus releasing the air confined in feed 2, and allowing the sewage to discharge into contact bed 2. This prevents any backing up in the distributing channel or settling tanks.

After standing for the required time, the sewage in the bed is discharged by a timed siphon, described later. As the sewage level in the bed falls, siphonic action is started in the withdraw siphon, and the compression chamber is drained. By this means, the compression dome and starting bell are vented, the blow-off trap is filled, and the chamber is ready for the next cycle of operation.

Timed Siphons.—These siphons, installed one to each contact bed, are for the purpose of automatically controlling the time the sewage stands in the bed. The general details of a typical timed siphon are shown in Fig. 195.

At the start, the main trap, blow-off trap and tile well in the timing chamber are filled with water. The size of the timing chamber and size of opening in the timing valve determine the period of contact and require adjustment after trial to obtain the specified period of contact. The timed siphon is controlled by the starting bell in the timing chamber, and until there is sufficient pressure in the starting bell to force the seal of the blow-off trap, the siphon will not discharge and the bed will stand full. The timing valve is located below the full water level in the contact bed, so that when the bed is full there is a continuous discharge through the timing valve into the timing chamber. The siphon receives air through the vent when the sewage has been drawn down to the low level and the discharge then ceases. While the timed siphon is operating, the draining siphon is discharging the sewage in the timing chamber, and at the end of the discharge the starting bell is vented, the timing chamber emp-

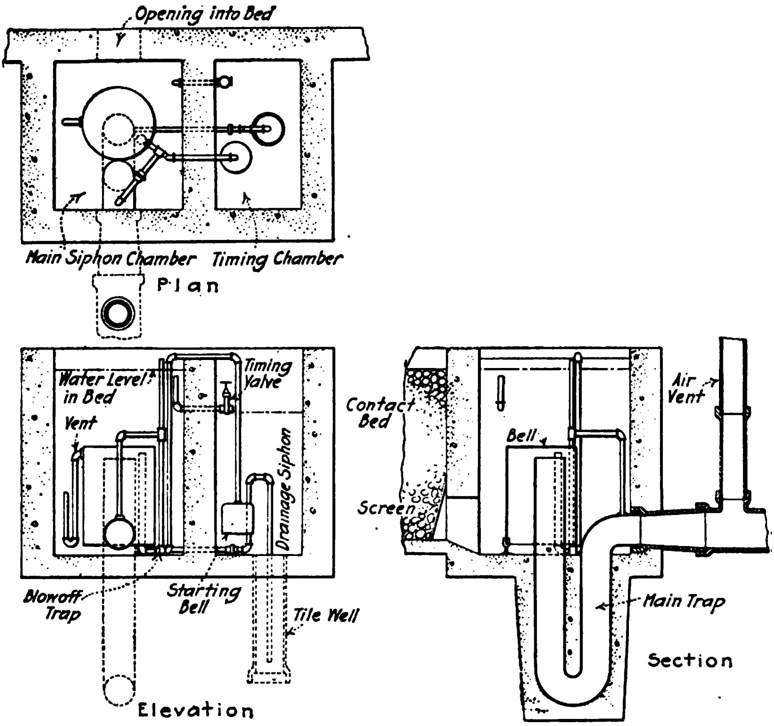


FIG. 195.—A timed siphon (Miller).

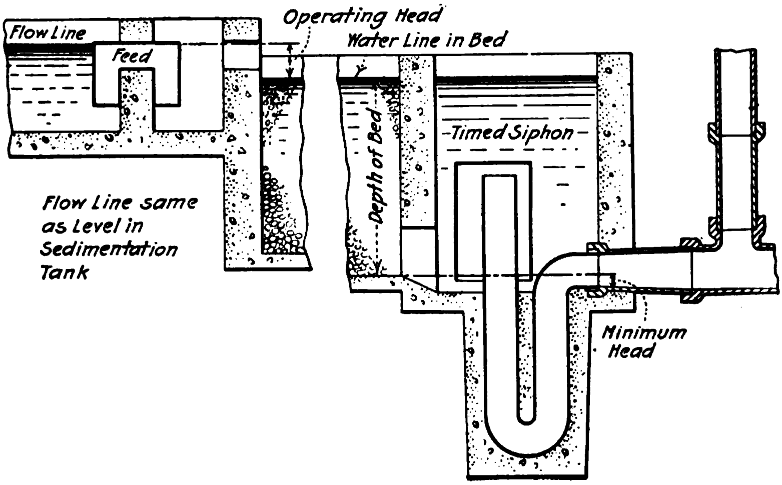


FIG. 196.—Loss of head with feeds and timed siphons.

tied, and the apparatus is ready for the next filling of the bed. A special type of "timed-siphon" is covered by letters patent No. 848696 granted to Shields, also No. 909340 for a method of making such siphons operate in rotation.

The air vent shown is not necessary where the siphon discharges into an open trough or where there is an outlet for the confined air not more than 50 ft. distant.

Loss of Head.—The loss of head required for contact bed apparatus is often a vital feature, and in this connection the figures given in Table 170, prepared by the Pacific Flush Tank Co., are useful. Fig. 196 indicates the dimensions referred to in the table.

TABLE 170.—SIZES OF FEEDS AND TIMED SIPHONS FOR VARIOUS RATES OF INFLOW

Maximum inflow, gal. per 24 hours	Size of Miller-Adams feeds required, in.	Operating head, in.	Depth of contact beds, ft.	Size of Miller timed siphons required, in.	Minimum head, Fig. 196, in.
5,000 250,000	6 × 10	9	3.5 to 4.0	5 to 6	2½
250,000 400,000	6 × 18	9	3.5 to 4.0	6 to 8	2½
400,000 750,000	8 × 18	11	4.0 to 5.0	8 to 10	2½
750,000 1,000,000	10 × 24	13	4.0 to 5.0	10 to 12	2½
1,000,000 2,000,000	12 × 30	15	4.0 to 5.5	12 to 14	2½
2,000,000 4,000,000	15 × 36	18	4.0 to 6.0	14 up	2½

Instead of building the filling apparatus at one end of the contact bed and the discharging apparatus at the other end, it is often more satisfactory to combine the two devices and locate all of the control apparatus for the entire plant together in a single building.

Air-flush Method of Control.—The Priestman-Beddoes air-flush control for siphonic apparatus works along somewhat different lines from those of the devices just described; Fig. 197 illustrates its operation. The upper half shows the inlet apparatus for one contact bed (left side) with connections to the inlet sluiceway of another bed (right side). The lower half shows two forms of discharge apparatus; the left side

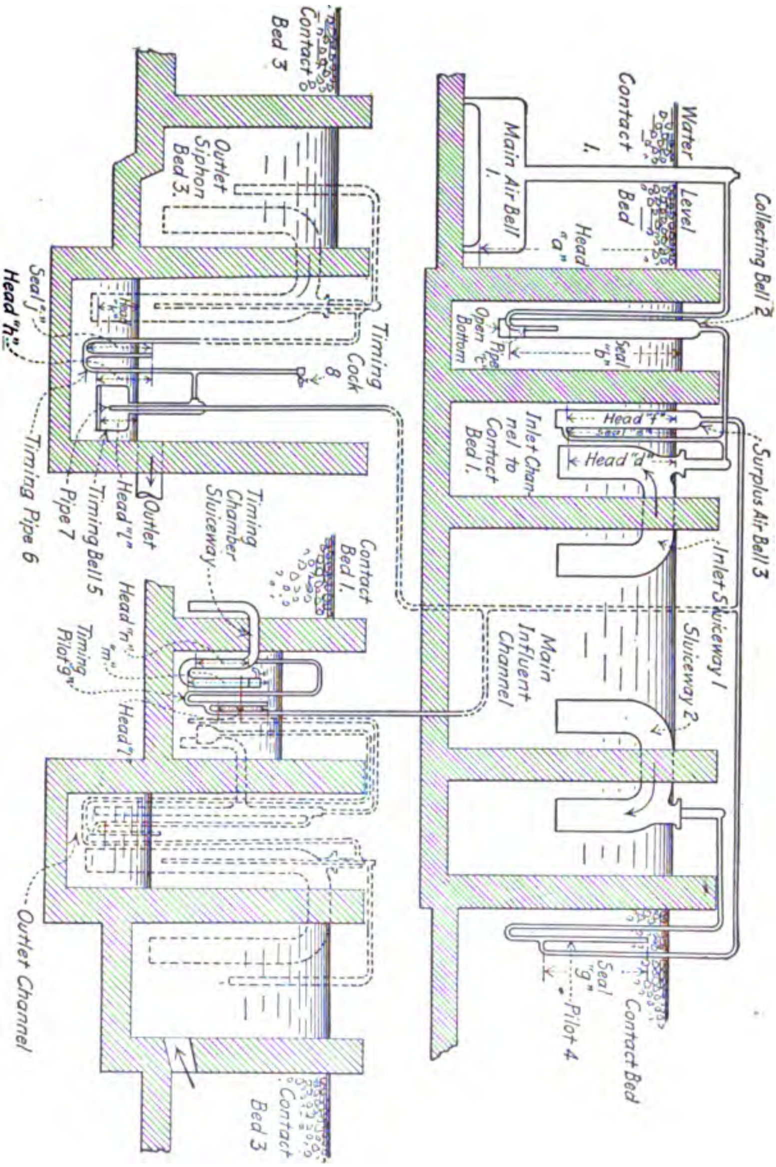


FIG. 197.—Priestman-Beddoes air-flush control for contact beds.

being a pneumatically timed siphon, and the right side a hydraulically timed siphon, both being connected to the inlet apparatus.

A main air bell is placed in the contact bed and collecting bell 2 in a sump which has a fixed water line during the filling of the bed. When the water in the bed has risen so that the head *a* is equal to the seal *b*, air is flushed into collecting bell 2. The pipe *c* is to insure the entire contents of bell 1 being discharged into bell 2.

Collecting bell 2 is piped to the top of the inlet sluiceway of the bed in which main air bell 1 is placed. After the air flush has taken place, the air from collecting bell 2 displaces the sewage from the inlet sluiceway until the head *d* equals the seal *e*, when the surplus air passes to the surplus air bell 3.

The pilot of the inlet sluiceway is placed in a sump that connects with the same bed as the inlet. When the surplus air from the closed inlet accumulates in bell 3 so that the head *f* equals the seal *g*, it ejects the water from the pilot, which releases the air from the sluiceway to be opened and allows the sewage to flow into the bed. As the bed fills, the sewage refills the pilot.

In the pneumatically timed siphon, air passes from surplus air bell 3 and displaces the water contents of timing bell 5 and the submerged part of timing pipe 6 until head *h*, equal to head *g*, is reached, when the supply to timing bell 5 is stopped by the blowing of inlet pilot 4. Pipe 7 is to prevent the air in bell 5 from escaping through inlet pilot 4 after the pilot has been blown. The head *h* gradually decreases as air escapes through timing cock 8. This reduces the seal *j* until it is equal to the head *k* of the outlet siphon, when this seal is blown, bringing the siphon into action.

In the hydraulically timed siphon, air passes from surplus air bell 3 and partly displaces the water contents of timing pilot 9 until the head *l*, equal to seal *g*, is reached, when the pressure is relieved by the blowing of inlet pilot 4. This reduces the seal *m* to less than the head *n* in the timing chamber sluiceway, which will allow pilot 9 to be blown and cause water to flow into the timing chamber, which will later operate the main outlet siphon as well as the timing chamber withdraw siphon.

Float Operated Valves.—The best known apparatus of this type is that installed by Snow & Barbour at Mansfield, Ohio. Fig. 198 shows a model of it. The Mansfield plant is made up of 5 contact beds, arranged as sections of a large pentagonal area, with the dosing apparatus in the center. Each section is dosed and emptied in rotation through separate pipes leading from each bed to the control apparatus. There are 2 cylindrical chambers, one for the influent and the other for the effluent. Each chamber is connected to the radiating pipes leading to each bed, and contains a cylindrical revolving gate, concentric with the chamber and carefully fitted to obtain water-tight

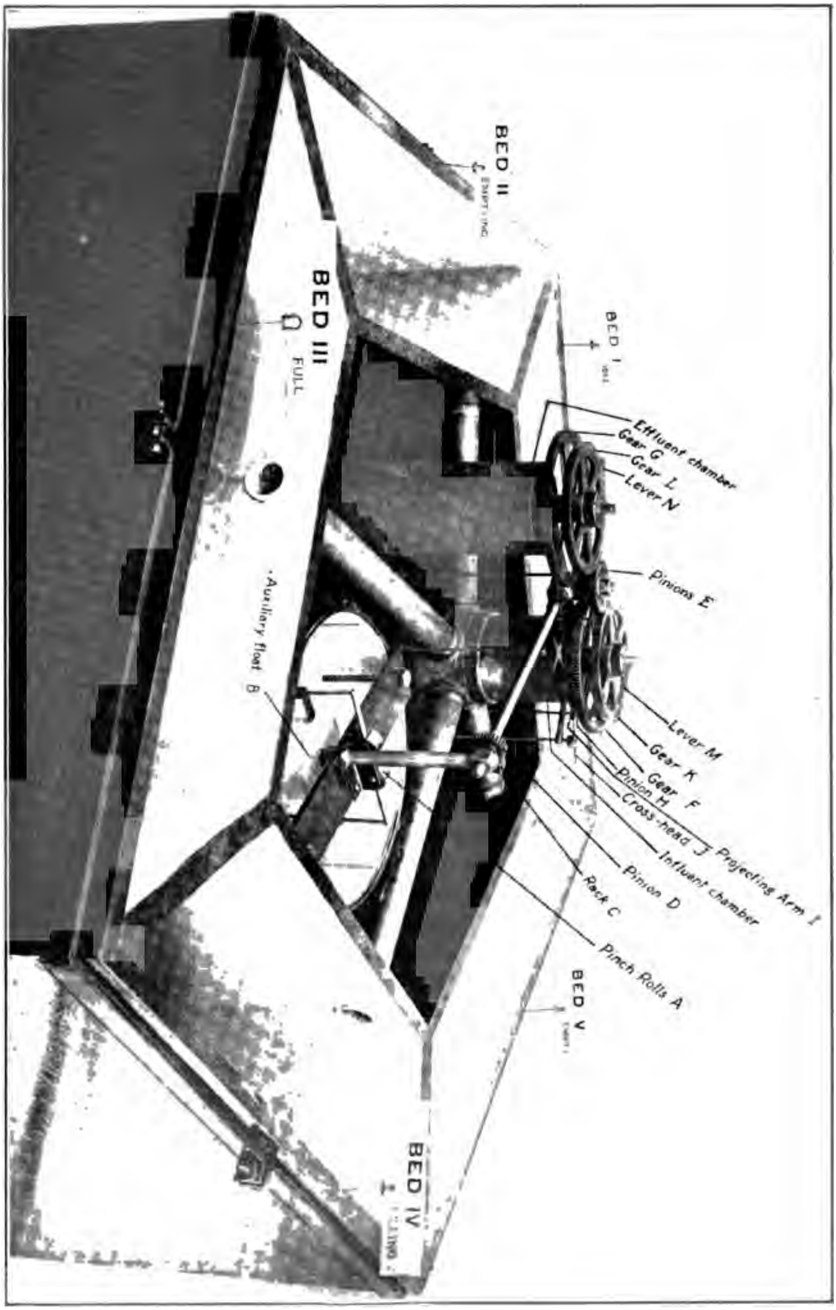


FIG. 198.—Model of Barbour dosing apparatus for contact bed.

joints. By revolving the cylindrical gate, its opening can be brought opposite each of the pipes leading to the beds, thereby shutting off the openings to the other beds. The power for revolving the gates is obtained from a float set in a separate compartment, which rises and falls quickly with the filling and emptying of each contact bed. Sewage enters this float compartment through a small auxiliary effluent pipe from the underdrain of the bed which is being filled, the pipes to the other beds being closed by valves also actuated by the float.

In Fig. 198, bed IV is being filled, bed III is standing full, bed II is emptying, bed I is out of commission, and bed V is standing empty ready for the next dose. When the plant was first put in operation it was customary to keep one bed out of commission for resting, and on that account, the apparatus was designed to permit the omission of any one bed in the dosing cycle.

Sewage enters the influent chamber from the settling tank, and flows through the gate opening and connecting pipe to bed IV. As the sewage level in bed IV rises, a small quantity of sewage floods back through the underdrain to the large float chamber, so that the sewage level in the float chamber rises with that in the bed. The large main float resting at the bottom of the float chamber (not shown in the illustration) tends to rise, but is held down by the friction of the pinch rolls *A* on the float rod. When, however, the sewage reaches a predetermined level near the top, the auxiliary float *B* is lifted and the pinch rolls *A* are turned, thereby releasing the float rod and allowing the main float to rise quickly. The upper end of the float rod contains the rack *C* which meshes with the pinion *D* attached by a friction clutch to the horizontal shaft. A friction clutch is used, in order that the float and rack may fall back without turning the horizontal shaft in the opposite direction. The horizontal shaft drives the bevel gears and pinions *E*. Over the influent and effluent chambers, and attached to the gate shafts by friction clutches, are the gears *F* and *G* respectively, which mesh with the lower pinion *E* and turn in a clockwise direction. The length of travel of the main float and rack is so regulated that gears *F* and *G* travel one-fifth of their circumference for one rise of the float. This movement turns the opening in the influent gate, shutting off the flow to bed IV and filling bed V. In a similar manner, the effluent gate turns, allowing the sewage in bed III to discharge while bed IV stands full. The rising of the main float opens a small valve in the bottom of the float chamber leading to the underdrain, and allows the sewage in the float chamber to discharge, whereupon the main and auxiliary floats drop back to their previous positions. At the same time that gear *F* turned, the blank pinion *H* underneath turned a similar amount, and the projecting arm *I* on this pinion engaged one of the arms of the cross-head *J*, fastened to the upper end of the valve stem controlling the back-flooding from bed

IV. This valve was given a quarter turn, thereby closing it, and at the same time another projecting arm on the same pinion *H* opened the small valve controlling the back-flooding from bed V. These small plug valves are so designed that each quarter turn of the stem opens or closes the valve.

When bed V is filled, the floats are raised as before and the train of gears is brought into motion in a similar manner to that previously described, except that the teeth of the mutilated gear *K* now come into mesh with the upper pinion *E*. The design is such that the influent

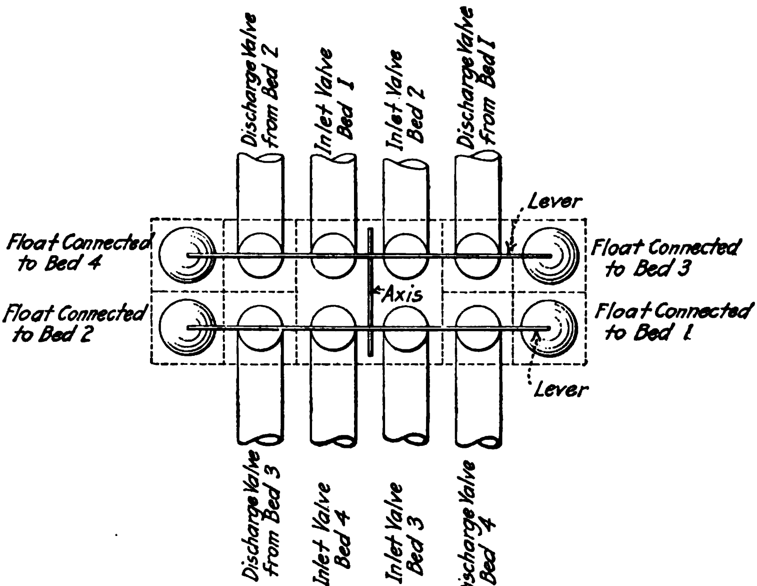


FIG. 199.—Connections of Cameron's dosing apparatus.

gate shaft, instead of being turned one-fifth of a revolution, is now turned two-fifths of a revolution. The influent gate shaft is allowed to move double the angle moved by gear *F* on account of the friction clutch in this gear. By this means the flow skips bed I, which is out of commission, and is turned into bed II. In a similar way, two operations later, mutilated gear *L* is brought into mesh with upper pinion *E* and the opening in the effluent gate skips from bed V to bed II.

It is possible to put any one of the beds out of commission by raising the levers *M* and *N* which hold the mutilated gears *K* and *L* in place, and turning the gears to the new positions, where they can again be locked by the levers. If all of the beds are in use, as is now (1915)

the case at Mansfield, the mutilated gears can be removed and each of the beds will be dosed in rotation.

The apparatus installed at Mansfield differs slightly from the model shown in that the pinch rolls *A* are replaced by a plug release which operates more satisfactorily in the larger machine. This apparatus is claimed to have advantages both as to cost and operation over air-controlled siphons where there are as many as eight or more beds to be dosed. This device, which is patented, was made for 5 beds by The

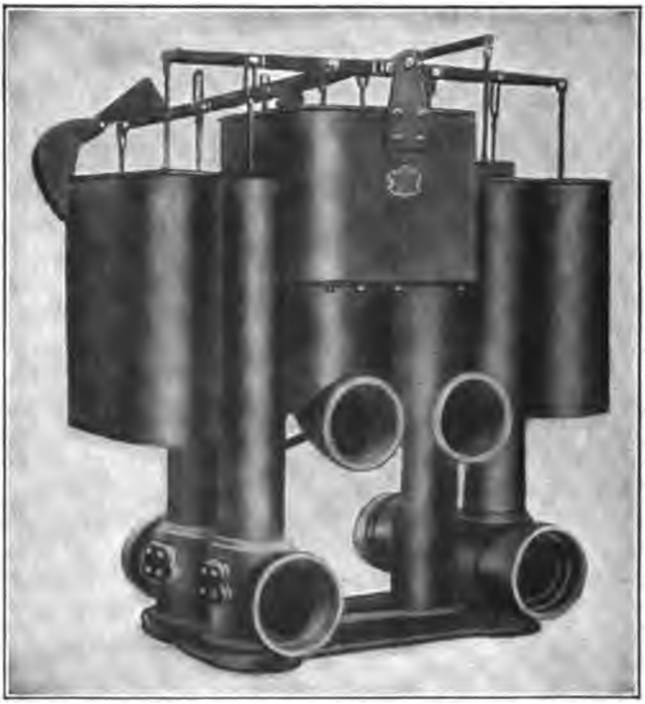


Fig. 200.—Alternating gear of Cameron's dosing apparatus.

Atlantic Works, Boston, Mass., at a cost in 1915 of $3850, including erection in the building on foundations previously constructed.

Cameron's Dosing Apparatus.—Among the English devices those designed by Cameron are prominent. Figs. 199 and 200 illustrate an installation for 4 contact beds. The stems of the inlet and discharge valves for beds 1 and 2 are connected to one lever and controlled by floats in beds 3 and 4. Likewise, the stems of the inlet and discharge valves for beds 3 and 4 are connected to the lever controlled by floats in beds 1 and 2. The inlet valve to one bed is on the opposite side of

the fulcrum from the discharge valve of the same bed. The ends of the levers are alternately raised and lowered by the floats, thus opening and closing the inlet and outlet valves of each contact bed in succession. While each bed is standing full in rotation, the float for that bed holds open the inlet valve of the next bed to be dosed and the outlet valve of the bed previously dosed, all the other valves remaining closed. The operation is well illustrated by the following table taken from Raikes' "Sewage Disposal Works," page 209.

When contact bed 1 is full,
Inlet 3 and outlet 4 opened by float 1 rising;
Inlet 1 and outlet 2 closed by float 4 sinking.
When contact bed 3 is full,
Inlet 2 and outlet 1 opened by float 3 rising;
Inlet 3 and outlet 4 closed by float 1 sinking.
When contact bed 2 is full,
Inlet 4 and outlet 3 opened by float 2 rising;
Inlet 2 and outlet 1 closed by float 3 sinking.
When contact bed 4 is full,
Inlet 1 and outlet 2 opened by float 4 rising;
Inlet 4 and outlet 3 closed by float 2 sinking.
The beds are dosed in the order 1, 3, 2 and 4.

Rocking-frame Control Apparatus.—The sewage treatment plant at the Soldiers' Home, Johnson City, Tenn., designed by Williams &

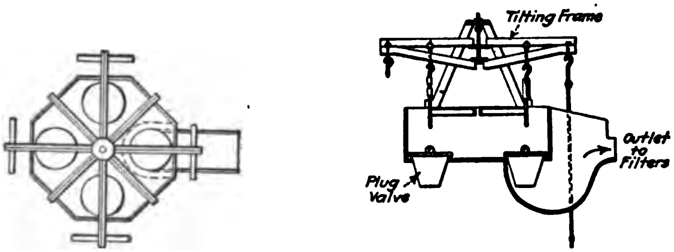


FIG. 201.—Rocking frame apparatus at Soldiers' Home, Johnson City, Tenn.

Whitman (*Engineering Record*, April 27, 1907, vol. lv), has an automatic dosing apparatus, Fig. 201, for controlling the flow of septic tank effluent to 4 contact beds.

The apparatus consists of a cast-iron box, octagonal in plan, in the bottom of which are 4 plug valves each connected to the distribution system of 1 bed. Over the center of the receiving box is a hook supporting the 4-arm rocking frame with each arm attached to the plug valve directly beneath it. When the rocking frame is in a horizontal plane all 4 valves are closed. By means of a weight on a swinging

arm, the plane of the rocking frame is thrown out of the horizontal in such a manner that the valve diagonally opposite the weight is lifted while the other three remain closed. When the bed which receives the effluent passing through the open valve is filled to a certain level, it overflows into a system of small pipes leading back into the controlling chamber. The overflow passes into a bucket hanging from the rocking arm diagonally opposite the bed which is to be filled next. The weight of the water in this bucket is sufficient to tilt the rocking frame, closing one valve and opening the next, and at the same time causing the swinging arm with the weight to pass through a horizontal angle of 90 deg. and settle in a position diagonally opposite the newly opened valve. When the water in the overflow bucket drains out through a small hole provided for this purpose, the weight on the swinging arm keeps the rocking frame tilted so that the valve opened by the weight of the water bucket stays open when the bucket is empty. When the second bed is filled to a certain level the next valve is opened in a similar manner and so on throughout the complete cycle.

Apparatus of a similar type has been installed at the Madison-Chatham sewage treatment plant in New Jersey, designed by Hering & Fuller, the apparatus being manufactured by the Ansonia Manufacturing Co.

The plant at the Soldiers' Home has a special apparatus for emptying each of the contact beds. It consists of a float in one bed connected by a system of levers to a valve in the bottom of an adjacent bed, so that when the float is raised, the valve is also lifted. When a bed has been filled, the next in order is always the one containing the float which is connected to the valve in the first bed. The float holds the valve open until the bed next in cycle has been filled and in filling has caused the second bed to begin to empty.

DOSING APPARATUS FOR TRICKLING FILTERS

The design of adequate dosing apparatus for trickling filters is closely allied with the design of the distribution system, described in Chapter XV. With the various forms of traveling distributors used extensively abroad no additional dosing apparatus is required so long as the sewage can be delivered to the distributor by gravity. In the United States, however, pressure nozzles have been used in preference to the traveling distributors, and the resulting complications necessary to obtain uniform distribution of the sewage have brought out a number of devices for controlling the flow. Some of these are modifications of methods used in sand filter or contact-bed installations, while others are entirely new.

The apparatus may be classified as follows: (*a*) siphonic apparatus;

(*b*) mechanically operated valves; (*c*) float operated valves, and (*d*) dosing tanks.

Siphonic Apparatus.—Automatic air-lock siphons, when used in conjunction with a specially designed dosing tank, are a satisfactory means of regulating the head on pressure nozzles. By varying the shape of the dosing tank it is possible to throw the spray from a nozzle over the surface to be sprinkled and accomplish uniform distribution of the sewage. Siphons used for this purpose are similar in design and operation to those employed to dose intermittent sand filters.

Size of Siphon.—The size of the siphon should be governed by the maximum rate of discharge of the nozzles or other distributing apparatus, and the allowable loss in head by friction in the siphon at that rate. Table 169 will be of some assistance in determining the required size. In general, the loss of head at the maximum rate of discharge should be definitely limited and for average conditions 0.5 ft. may be all that can be allowed.

In making such computations, allowance should be made for losses from the following sources: (1) friction in equivalent length of straight pipe due to roughness of surface; (2) head lost at entrance; (3) head lost in changes of direction or bends; (4) head lost in sudden enlargement of the section; (5) head lost in sudden contraction of the section; (6) head lost at obstructions such as valves, etc.; (7) excessive surging due to impact of sudden discharge.

The head lost in entrance may be materially reduced by using the principle of rounded orifices and bellmouth openings. Sudden changes in sectional area should be avoided so far as possible and where a change in section is necessary it should be a gradual one. Reference to Volume I and to the special works on hydraulics there mentioned will furnish the desired data for estimating these losses.

Fitchburg Dosing Apparatus.—The sewage treatment works at Fitchburg, Mass., are designed to care for not only the normal daily sewage flows but also excessive rates of flow during storms when considerable surface water is admitted to the sewers. The dosing apparatus is designed to shut off the inflow to one of the 2 dosing tanks while that siphon is discharging and allow the other tank to be filled, thus assuring the same size of dose and the same distribution of the sewage on the filter at each dose regardless of the rate of sewage flow, until a maximum storm-water rate is reached at which one of the tanks will go into continuous operation. By using 2 dosing tanks designed to operate alternately, the point at which continuous operation begins is kept higher.

This apparatus is illustrated in Fig. 202. The siphons and sluiceway were designed and installed by the Merritt Hydraulics Co., under

plans prepared by D. A. Hartwell, Chief Engineer, and Harrison P. Eddy, Consulting Engineer, of the Sewage Disposal Commission.

Mechanically Operated Valves.—In connection with the report of the Board of Advisory Engineers to the Sewerage Commission of Baltimore in 1906, Friederic P. Stearns suggested the use of a butterfly valve driven by cams to produce a specific variation in head on pressure nozzles, as a means of obtaining satisfactory distribution of sewage on the surface of the trickling filters. This scheme has since been worked out and adopted.

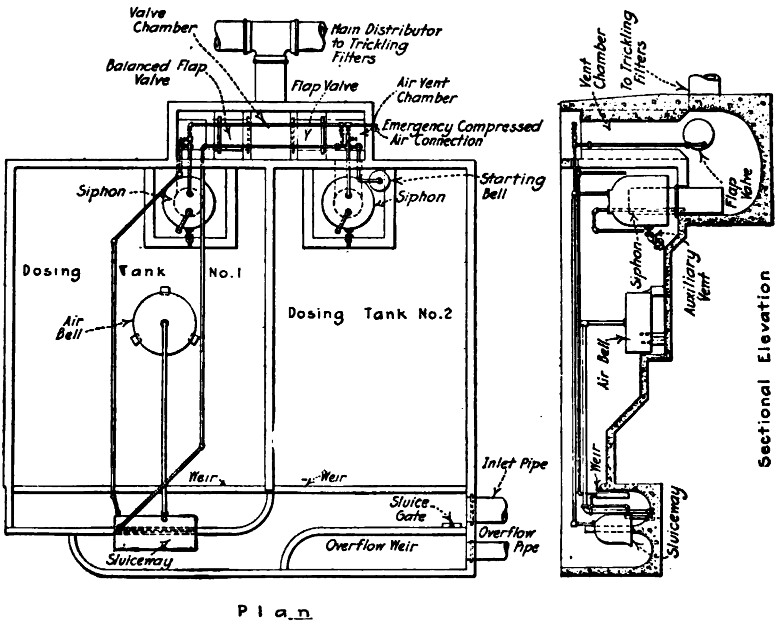


FIG. 202.—Dosing tanks and siphonic apparatus, Fitchburg, Mass.

Taylor's Undulating Valve.—Investigations at Waterbury, Conn., reported by W. G. Taylor in *Engineering News*, November 11, 1909, showed that it was possible to obtain a substantially uniform distribution of sewage by means of an undulating valve designed to vary the head on the nozzles between a maximum and a minimum. A typical section of the valve, as proposed, is shown in Fig. 203.

"The new valve consists essentially of a cylindrical or rectangular throat within which is placed a rotable valve platen having such specific sectional contour that it will produce as a result of its rotation, a variation in pressure at the outlet port, which will precisely follow a predetermined cyclic pressure curve. The actuating apparatus consists essentially of a small low-speed

standard motor with a single train of encased reduction gears all of which may be attached by brackets to the valve head or mounted on an independent pedestal. So long as the valve is rotated a true cyclic pressure variation must be followed and the influent applied at the correct rate. Variation in the speed of rotation does not modify the effective filter rate or change the form of pressure curve but simply alters the length of the dosing cycle. On each of the vertical side faces of the valve is placed a circular valve head and the removal of either will permit an examination of the interior or removal of the valve platen."

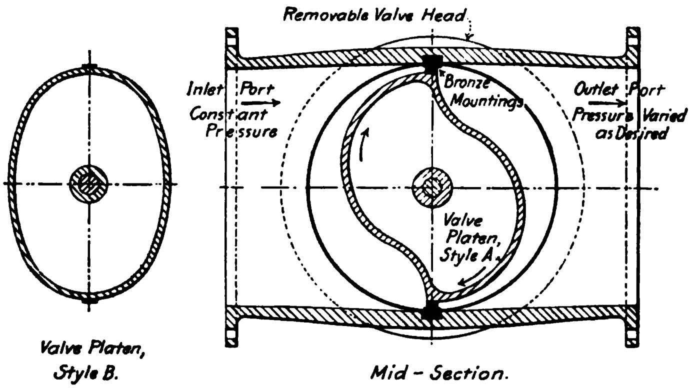


FIG. 203.—Taylor undulating valve.

Baltimore Butterfly Valves.—The following description and Figs. 204 and 205 illustrating the chief features of the control apparatus were furnished by Calvin W. Hendrick, Chief Engineer of the Baltimore Sewerage Commission:

The trickling filters at the Back river disposal plant are dosed by nozzles which spray the sewage over the surface of the stone. The head on the nozzles varies from 9 or 10 ft. to nearly zero.

In the control house the sewage passes first into a "constant head chamber" where the sewage level is maintained at a constant elevation by means of large float-operated shear gates of the ball-cock type, which admit sewage from the outlet channel leading from the tanks. The floating balls for these valves are about 3 ft. 6 in. in diameter and are mounted upon rods extending outward from the gates and pivoted upon arms forming part of the gate seat castings.

Leading from this chamber are ten 18 × 27-in. rectangular conduit castings (one to each of the 10 filter beds) through which the sewage passes on its way to the nozzles. The opening to each of these castings is controlled by an electrically operated sluice gate. Each motor is connected to an auto-starter actuated by a trip on a float rod extending upward from a float in the outlet channel. The trips on these rods are set at four different levels

so as to bring sections of the trickling filters into operation one after the other by opening the motor-operated valves. For example, if 2 units are in operation and sewage begins to back up in the outlet channel faster than it can be taken care of by the area of trickling filters then in service, the float under the third auto-starter is raised sufficiently to trip the latter and open up a sluice gate on the inlet line to section 3 of the trickling filters. The

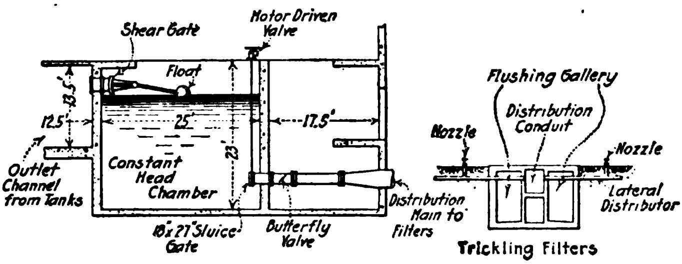


FIG. 204.—Sectional elevation of dosing apparatus, Baltimore.

power for operating these electrically controlled sluice gates is obtained from the hydro-electric plant on the outfall sewer line.

Inside of each rectangular casting above referred to is a flat iron plate swung on its horizontal axis by a shaft extending through a stuffing box to the outside of the casting. This plate fits the inside dimensions of the casting with a clearance of about ½ in. The end of the shaft is geared to a 1-h.p. direct-current motor. The plate is turned through 1 revolution

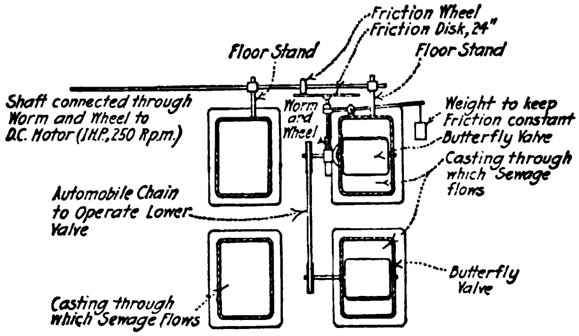


FIG. 205.—Operating device for butterfly valve, Baltimore.

(2 cycles) approximately every 7 minutes, thus alternately damming and releasing the sewage flow to the nozzles. The spray on the filters under this action falls first on the stone immediately adjacent to the nozzle, gradually increasing its breadth to a distance of 7½ ft. away from the nozzle, at which moment the flat plate in the casting has turned till it is in a horizontal position and the maximum amount of sewage is flowing. Fig. 204 is a sectional elevation of the constant-head chamber and shows the relative

location of the float-operated shear gates, sluice gates and butterfly valves; also the relative elevations of the filter nozzles. Fig. 205 is a front elevation of the butterfly valves showing 4 valves, two above and two below. The driving mechanism for two of these valves only is indicated.

Pennypack Creek Butterfly Valves.—Between the Imhoff tanks and the trickling filters at the Pennypack Creek plant at Philadelphia is an equalizing tank which serves to maintain a constant head on a butterfly valve set in the pipe line between the equalizing tank and the trickling filters.

"The effective nozzle pressure and, therefore, the spray from the nozzles of the percolating filters is controlled by the action of a butterfly valve in the feed line between the constant head tank and the filters.

"This valve consists of a circular bronze leaf of the diameter of the pipe in which it operates, pivoted centrally on a vertical brass shaft. It has no seat and is prevented from making a complete revolution by exterior stops at full open and closed positions. The bottom of the shaft turns in a socket cast in the pipe section and drilled to fit the shaft. At the top the shaft passes through an ordinary stuffing box and terminates at a bevel gear.

"The weight of the shaft and the leaf is taken by a ball-bearing beneath the gear, resting on a frame which is adjustable vertically to accurately center the leaf within the pipe. A small by-pass, controlled by a hand valve, is connected around the leaf to maintain the water level in the filters during a prolonged shutdown of the butterfly valve.

"The bevel gear on the vertical shaft is in mesh with another one on a horizontal shaft crossing above the valve, carried in boxes supported on brackets which are bolted directly to the pipe section. To this horizontal shaft is attached the lever for operating the valve.

"The bronze leaf has no sharp edges and the surfaces are curved to oppose a minimum resistance to the flow. It is absolutely balanced and will maintain itself in any position.

"The action of the butterfly valve is controlled by a machine driven by the sewage (Fig. 206). The constant head tank is tapped by a pipe which conveys a small amount of clarified sewage to the machine, where it passes through a combination impulse and gravity water-wheel and is then deposited on the surface of the percolating filters. This water-wheel transmits its motion to a shaft which drives the main shaft of the machine by means of a worm and worm wheel. An emergency drive is provided in an electric motor.

"Both water-wheel and motor are connected to the driving shaft by centrifugal friction clutches which grip when a certain speed is attained, so that each attains considerable power before being called on to do any work. The motor is thrown into service automatically when the water-wheel stops or slows down from any cause, and, as the stopping or slowing down of either one releases its friction clutch, the one in service does not have to carry the inactive one with it.

"The main shaft has 3 cams, one of which is fixed and two are loose on the shaft. The loose cams have spring clutches which engage with the

shaft and are held open by arms actuated by electro-magnets, which in turn are controlled by float switches at the constant-head tank. These cams transmit a motion, in accordance with their particular outlines, to a lever which is connected directly to the operating shaft of the butterfly valve.

"The cams are so shaped as to give a uniform distribution of the spray from the nozzles to full play and return. The difference in the shape of the 2 cams is such that one has a longer period of spray at full head than the other, and, therefore, is used when sewage reaches the works at times of maximum flow.

"The fixed cam, with its lever, operates a counterweight which closes the valve in comformity with the return side of the cam.

"The electro-magnets are connected with two-way gravity-throw switches at the constant-head tank, operated by a ball float and so adjusted as to maintain a constant water level by keeping in service the cam which is properly adapted to the rate of incoming sewage. When no sewage is entering the tank, the machine is running idle, the cam grips being held open by the magnet arms. As sewage enters and the water level tends to rise in the constant-head tank, the float causes the magnets to operate and the short-play cam comes into action. If this is not sufficient to care for the quantity, the large cam is thrown in by the magnets. If the level continues to rise, an alarm gong is rung, giving notice to the attendant that additional filter-bed capacity is required. When the water-level falls, the operations are reversed until the machine is again running idle.

"The magnets can be operated either from the line or a storage battery, a mechanical make-and-break contact being provided on the main shaft, so that current is sent through the circuit for an instant only during each revolution of the cams.

"An adjustment on the valve-operating lever provides the varying opening of the butterfly to compensate for the changing filter area and maintains the radius of action of the spray nozzles under all conditions.

"The machine is ball-bearing throughout and requires about 1/16 h.p. to operate." (Report of Bureau of Surveys, 1912, page 253.)

Reading Butterfly Valves.—The first unit of trickling filters constructed at Reading, Pa., in 1907 was dosed for some time by float-operated butterfly valves designed and patented by O. M. Weand.

"The dosing apparatus is located between the main from the septic tank and the filter bed, midway of the side of the latter. It consists of an elevated wooden tank 19 ft. in diameter, divided by interior walls into 3 compartments of unequal size, the flow into and out of which is controlled by 3 butterfly valves operated by floats. The largest compartment is called the dosing compartment, the next in size the storage compartment and the smallest the overflow compartment. A 30-in. riveted-steel pipe connects with each compartment through the bottom of the tank. The sewage coming from the main from the septic tank flows through a 30-in. pipe at right angles to the former and then into another 30-in. pipe parallel to the side of the filter bed. This pipe terminates in the storage compartment, which is always open to the flow from the septic tank. Two butterfly

valves are placed between the pipe leading to the storage compartment and a parallel one to the dosing compartment. A third butterfly valve is interposed between this latter line and the main distributor to the filter, the latter being likewise connected to the overflow compartment by a short length of pipe. Cast-iron riser pipes, containing the floats which operate the valves, are connected by 8-in. nipples to the 30-in. pipe leading to the dosing tank.

Fig. 206.—Operating machine, Pennypack Creek disposal works, Philadelphia.

All of the piping is $\frac{3}{16}$-in. riveted steel, except the float pipes. The valves are 20 in. in diameter.

"The operation of the apparatus is as follows, the valves being described by the letters on the diagram, Fig. 207. Starting at a time when there is no flow from the nozzles, valve *A* is open and *B* and *C* are closed. The sewage is then filling both the dosing and storage compartments, the level being the same in both, since they are connected through valve *A*. As the water

rises, valve *B* gradually opens, and when the compartments are full, equal to a head of 7 ft. on the nozzles, *B* is wide open, *A* closes and *C* opens, allowing the compartments to drain onto the beds and also the sewage to flow through valve *B* directly from the septic tank. When the head on the nozzles has dropped to 3½ ft., valve *B* closes, shutting off all connection between the septic tank and the nozzles and allowing the storage compartment to refill. The discharge from the dosing compartment continues until the head on the nozzles is 2 ft., when valve *C* closes and *A* is opened, restoring the starting condition. A weir is provided between the dosing and overflow compartments to allow a discharge in case the sewage rises over 7 ft. above the nozzles. The cycle of operations described will take from 7 to 8 minutes, the interval of rest being from 2 to 4 minutes of this period." (*Engineering Record*, vol. lvi, Oct. 5, 1907.)

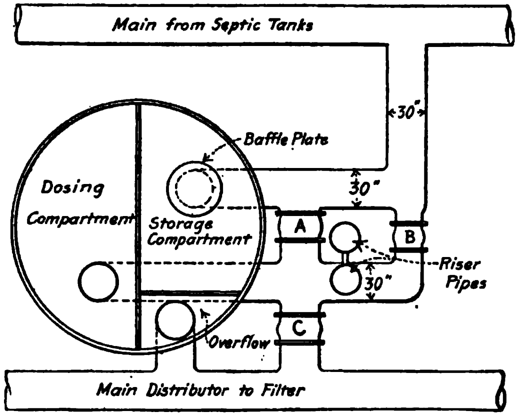


FIG. 207.—Arrangement of dosing apparatus, Reading, Pa.

Washington Controlling Device.—Provision was made in the sewage treatment plant at Washington, Pa., installed in 1908 from the designs of R. W. Pratt, for controlling the flow from the septic tanks to the trickling filters so that there will always be sufficient head to operate the nozzles. In Fig. 208 is shown a plan and sections of the control apparatus.

In a small chamber constructed at one end of the effluent channel from the septic tanks is a butterfly valve swung by a float located in a separate compartment of the chamber. When the head on the nozzles becomes low, the depth of sewage in the float chamber has dropped to such a level that the butterfly valve is closed by the float and the discharge from the septic tanks ceases. This stores the sewage in the septic tanks until the level rises approximately 3, 6, 12 or 24 in., as may be desired. When the desired level is reached a small stream of sewage is siphoned out through one of the 3-in. pipe siphons into the float-chamber.

As this chamber fills, the float rises and opens the butterfly valve, allowing the sewage to flow out under an increased head. When the stored-up sewage has been discharged, the float-chamber drains, the valve is closed and the process is repeated. (*Engineering News*, vol. lx, July 16, 1908.)

DOSING TANKS

A study of the action of pressure nozzles in trickling filters has shown that with the forms of nozzles thus far placed on the market and with a pressure head not over 10 ft., it is impossible to obtain a uniform distribution of sewage over the filter surface if the nozzles operate under a constant head. If, however, the head is made to fluctuate from a maximum of 5 to 10 ft., down to 1 or 2 ft. the spray is thrown in and out over

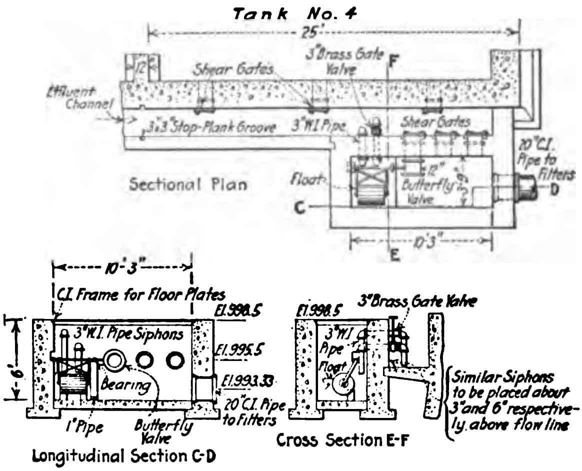


FIG. 208.—Controlling apparatus at Washington, Pa.

the surface, and by regulating the variation in head according to a predetermined plan, practically uniform distribution may be accomplished. While this variation in the pressure may be obtained by hand-operated valves, as has been done at Columbus, Ohio, it has been found much more satisfactory to use a tank or other device designed to work automatically.

Special forms of dosing tanks have been installed at a number of plants. The form usually adopted is a hopper shape or is tapered by either sloping side walls or steps, so that the capacity of the upper portion of the tank is much greater than that of the lower portion. This causes the nozzles to discharge longer under the higher heads and throws a larger proportion of the spray to the larger outer areas. A form of

tapered dosing tank used in conjunction with an automatic siphon is covered by letters patent controlled by the Merritt Hydraulics Co.

The exact shape of the dosing tank to accomplish uniform distribution with a given type of nozzle is largely a matter of experiment. There are a number of features entering into the design of a large tank which may not affect a small experimental tank. The following points should be considered in designing a large tank, and proper allowances made for each:

1. The capacity for each foot in depth required to give uniform distribution of spray.
2. While the siphon is getting under way, the nozzles discharge under heads from zero to maximum head. The quantity thus discharged should be deducted from the respective volumes required at each foot of depth.
3. While the siphon is getting under way, the nozzles discharge under heads from zero to the maximum, thereby reducing the capacity of the dosing tank, drawing down the high-water level and resulting in a lower maximum head on the nozzles than would have been the case if the nozzle discharge did not lag behind the siphon discharge. This is largely due to the inertia of the great mass of water, which requires time to attain the maximum velocity.
4. Provide additional capacity above the maximum flow line for the capacity lost in filling the air vent or other spaces below the hydraulic grade line. The entire distribution system, so far as possible, should be below the minimum hydraulic grade line.
5. After the siphon sniffs, the nozzles will continue to discharge for a short period. This will affect the distribution at low heads.
6. Set the high-water line at the proper elevation to compensate for friction losses in the siphon, distribution system and appurtenances.
7. The maximum high-water level has only slight effect, as it is immediately reduced by the draft from the tank.
8. When the siphon is discharging at low heads, the friction losses are greatly reduced and consequently more of the head is available at the nozzles.
9. If sewage is allowed to enter the tank continuously during discharge. the capacity of the tank at different depths and the distribution are affected according to the rate of sewage flow.

The majority of plants are provided with but a single dosing tank or a tank for each unit or group of filters. There is some advantage in two dosing tanks, each discharging to the same filter area, as is the case at Fitchburg, Mass. If provision is made to divert the inflow from one tank to the other during the discharge of the first, it will be possible to control the distribution and the size of the dose so as to be the same at all times during normal rates of sewage flow. The rate at which continuous operation of the siphons will begin will be materially increased by the use of 2 tanks, which is advantageous where high rates of flow above the normal must be cared for.

CHAPTER XIX

DISINFECTION OF SEWAGE AND SEWAGE EFFLUENTS[1]

The term "disinfection" is used here to designate the treatment of sewage or infected water so as to reduce greatly the number of bacteria in it. Incidentally this treatment of sewage may deodorize the liquid more or less, and prevent or retard putrefaction. By "sterilization" is meant the destruction of all organisms. In the methods of treating sewage previously described a change in the character of the organic matter originally present is the main object sought, whereas in disinfection and sterilization the main object is to kill the bacteria in the liquids treated.

Deodorizing.—A notable early use of disinfectants with sewage was for deodorization rather than disinfection, at London from 1884 to 1890. It was described by W. J. Dibdin as follows:

"When I received the order to deodorize the London sewage, prior to its discharge into the Thames at Barking Creek and Crossness, the only material available in any quantity was chloride of lime. The use of this material produced apparently good results at first, but when the effect of the chlorine disappeared, the putrefaction of the sewage matters was objectionable in the highest degree, being worse than the nuisance from untreated sewage. I concluded that we must apply a deodorant which would supply oxygen without acting as a germicide. Permanganates were used . . . The nuisance disappeared in consequence of the oxidizing action of the permanganate, which also allowed the aerobic organisms to purify the river while precluding the putrefactive anaerobic action, which followed the use of chloride of lime." (*Jour. Assoc. Eng. Socs.*, 1908, vol. xi, page 310.)

The trouble existing in the Thames was due to deficient dissolved oxygen, and calcium hypochlorite did not furnish enough oxygen to alter the conditions. It simply killed the bacteria in the sewage, which immediately underwent decomposition when discharged into the river, then teeming with bacteria. The addition of permanganates to the sewage probably furnished a moderate supply of oxygen, sufficient to improve the river conditions.

[1] This chapter has been prepared with the assistance of Julius W. Bugbee, Chemist in Charge of the Sewage Purification Works of Providence, R. I. The material relating to market conditions and the manufacture of bleaching powder was furnished by Martin L. Griffin, Chemist to the Oxford Paper Co., Rumford, Me.

Electrolytic Processes.—A number of proprietary processes have been promoted to deodorize, and in some cases sterilize, sewage and industrial wastes. Among these may be mentioned the Webster process, installed experimentally at Crossness, England, to treat London sewage, in 1889. The raw sewage flowed, in contact with iron electrodes, through long troughs, a current of about 2 volts and 0.9 ampere per square foot of electrode being used. For the treatment of crude sewage it was estimated that 240 lb. of iron and 450 kw.-hr. of electricity would be consumed per 1,000,000 gal. treated. This process was essentially treatment by chemical precipitation, as the iron was dissolved from the electrodes and precipitated as hydrate of iron together with the suspended matter of the sewage. However, Webster recognized that the hypochlorites formed by the electrolytic action upon salts in the sewage had a disinfecting value. A plant was installed in 1890 at Bradford, England, and it is reported that 70 per cent. of the putrescible organic matter of the sewage was removed by this process.

A process was invented by Albert E. Woolf, of New York, by which a strong brine was electrolyzed, resulting in the production of chlorine and caustic soda, which were allowed to recombine in the form of sodium hypochlorite. In the spring of 1893 the sewage of about 30 dwellings at Brewster, N. Y., was treated by the addition of hypochlorite solution made in this way, under the direction of the Health Department of New York City. (*Engineering News*, 1893, vol. xxx, page 41.) This village was situated on a small stream discharging into Croton Lake. The object of the treatment at Brewster was to protect the water supply of the City of New York. For every million gallons of sewage treated 1600 lb. of salt were used. The plant required an electric current of 700 amperes at 5 volts. "This seems to have been the first plant established for the specific purpose of destroying bacteria. Before that time the removal of the organic matter had been the aim." (Water Supply Paper 229, page 27, U. S. Geol. Survey.) The original plant was burned in 1911 and a new plant using a chlorination process was built to treat the sewage of most of the town, instead of the few houses for which the first works were installed. Results obtained with the second plant are given later in this chapter.

REASONS FOR DISINFECTING SEWAGE

The primary object of disinfection is usually to reduce to a negligible degree the danger of the spreading of disease by pathogenic germs. It is unusual for a water receiving sewage or sewage effluents to be used for domestic consumption without previous treatment, and with modern facilities for purifying and disinfecting a water supply, it is generally wiser to apply the corrective measures to the water supply than to the

sewage. There will doubtless be some conditions, however, under which it will be desirable to disinfect the sewage, even though the water supply be effectively treated. Conditions will rarely justify the use for domestic purposes of an unpurified water which has been recently contaminated with sewage, even though the latter has been purified and disinfected as thoroughly as may be practicable by methods available. The danger of contracting typhoid fever by eating sewage-contaminated oysters, discussed on page 123, is sufficient to justify the disinfection of sewage of some communities, and several plants for this purpose are now in operation, notably that at Providence, R. I.

In some places where the water at bathing beaches is likely to be contaminated by local discharge of sewage, it may be practicable to reduce the danger of infection by disinfecting the sewage before its discharge.

It often happens that offensive conditions are caused by insufficient diluting capacity of the stream into which sewage is discharged. Occasionally such streams flow into larger rivers or the ocean within relatively short distances and thereafter ample dilution is afforded the sewage. Under such circumstances it is conceivable that disinfection of the sewage before its discharge might so retard bacterial activity as to permit the escape of the sewage-laden waters of the stream into the larger diluting waters, before offensive conditions could become established. While disinfection may prove of value in such cases, the opportunities for its successful application for this purpose will probably be of rare occurrence and, on account of natural growth in the population contributing sewage, will generally be of short duration.

Bacterial Content of Sewage and Effluents.—The number of bacteria in sewage depends upon its strength, age and character. It varies from 1,000,000 per cubic centimeter in sewage which is fresh, dilute or affected by the presence of considerable acid or other antiseptic industrial wastes, to perhaps 20,000,000 in very strong sewage or in sewage of moderate strength which is sufficiently old to have permitted a large development of bacteria. The pathogenic character of some sewage-carried bacteria was discussed in Chapter III.

The relative bacterial efficiencies of strainers, tanks, sand filters, contact beds, trickling filters and secondary filters are shown in Table 171, from a report by Clark and Gage. (Mass. State Board of Health, 1910, page 269.)

The table is made up of averages, and individual samples frequently contained many more than the average number of bacteria. These results were obtained from experimental filters operated under most favorable conditions and expert supervision. It is to be expected that large plants, operating under practical working conditions, will produce somewhat inferior results.

TABLE 171.—REMOVAL OF BACTERIA BY VARIOUS PURIFICATION SYSTEMS.—(*Continued*)

(Results of experiments at Lawrence)

Filter number	Filter				Aver. rate of application of sewage, 1910, gal per. acre per day	Character of sewage applied	Bacteria per cubic centimeter			Per cent. of bacteria removed		
	Dimensions		Material				20°C.	40°C.		20°C.	40°C.	
	Area	Depth	Kind	Size				Total	Red		Total	Red
Effluent, Trickling Filters												
135	2.18 sq. ft.	10′ 0″	Brok. stone	¼″ to 1″	2,000,000	Settled sewage	22,300	4,030	2,970	98.40	98.42	98.58
136	2.18 sq. ft.	10′ 0″	Brok. stone	¼″ to 1″	2,000,000	Settled sewage	70,800	10,100	8,300	94.91	96.04	96.01
222[6]	0.005 acre	7′ 6″	Brok. stone	¾″ to 1½″	1,500,000	Andover settled sewage	256,000	22,800	20,000	82.50	93.70	92.60
248	2.18 sq. ft.	8′ 0″	Brok. stone	¼″ to 1″	2,000,000	Settled sewage	287,800	43,900	33,800	79.30	82.80	83.75
360A	....	....	....	....	....	Settled sewage	206,700	20,200	15,200	85.10	92.08	92.69
360B	....	....	....	....	....	Settled sewage.......	141,800	16,100	12,300	89.80	93.69	94.09
360C	....	....	....	....	....	Settled sewage	158,300	13,000	9,700	88.70	94.90	95.35
360 Entire	4.36 sq. ft.	8′ 9″	Brok. stone	1″ to 2″[7]	1,500,000	Settled sewage	169,400	17,700	13,300	87.90	93.06	93.60
361—2 ft.	....	....	....	....	....	Settled sewage	406,800	41,700	32,400	70.70	83.70	84.40
361—4 ft.	....	....	....	....	....	Settled sewage	287,300	26,000	20,100	79.40	89.80	90.34
361—6 ft.	....	....	....	....	....	Settled sewage	254,500	18,200	13,700	81.70	92.85	93.42
361 Outlet	4.36 sq. ft.	8′ 9″	Brok. stone	1″ to 2″[7]	1,500,000	Settled sewage	198,100	20,000	15,400	85.80	92.15	92.59
362	4.36 sq. ft.	8′ 9″	H'd. clinker	4″ to 6″[7]	1,500,000	Settled sewage	144,800	22,700	18,600	89.60	91.10	91.05
391	0.54 sq. ft.	8′ 0″	Brok. stone	½″ to 1½″	1,000,000	Settled sewage	509,200	78,400	50,800	63.40	69.40	75.60
392[6]	0.54 sq. ft.	8′ 0″	Brok. stone	½″ to 1½″	1,000,000	Andover settled sewage	288,800	23,400	17,300	80.20	93.50	93.60
Effluent, Secondary Filters												
363	2.18 sq. ft.	4′ 6″	Met. coke	Pea size	4,000,000	Effluent, Nos. 135–136	26,500	2,500	2,000	72.00	60.30	48.70
371	0.54 sq. ft.	2′ 0″	Sand	0.23 mm.[1]	10,000,000	Effluent, Nos. 361–362 after settling	447,900	10,900	7,850	27.40	46.80	48.40

[1] Effective size. [2] Mixed coarse and fine sand. [3] 100 per cent. less than 1 in., 75 per cent. less than ½ in., 0 per cent. less than ¼ in.
[4] Exposed surface available, 314 sq. in. per gallon of content. [5] Exposed surface available, 187 sq. in. per gallon of content. [6] At Andover.
[7] Mean diameter between these limits. [8] Roofing slate.

TABLE 171.—REMOVAL OF BACTERIA BY VARIOUS PURIFICATION SYSTEMS

(Results of experiments at Lawrence)

							Bacteria per cubic centimeter			Per cent. of bacteria removed		
							20°C.	40°C. Total	40°C. Red	20°C.	40°C. Total	40°C. Red
Lawrence street sewage (samples taken from sewer)							1,507,300	400,700	302,600	......	......	.....
Regular sewage (as received from force main at station)							2,095,600	448,100	363,600	......	......	.....
Settled sewage							1,386,300	254,900	207,900	33.80	43.00	42.90
Effluent, strainer E (12-in. depth buckwheat coal, rate 800,000 gal. per acre per day)							874,200	141,900	115,800	58.50	68.40	68.20
Andover, regular sewage (town sewage before settling)							3,476,100	646,500	539,200	......	......	....
Andover settled sewage (tank capacity 13,500 gal.—average storage = 2 hours)							1,461,800	360,600	271,400	58.00	44.30	49.70
Fresh sewage (from toilet room at station)							3,241,600	597,700	553,000	......	......	.....
Effluent, Imhoff tank							1,730,000	343,200	189,200	46.60	42.60	65.80
Effluent, Sand Filters												
Filter number	Dimensions: Area	Dimensions: Depth	Material: Kind	Material: Size	Aver. rate of application of sewage, 1910, gal. per acre per day	Character of sewage applied						
1	0.005 acre	5' 0"	Sand	0.48 mm.[1]	50,000	Regular sewage	6,200	4,300	3,800	99.71	90 04	98.96
2	0.005 acre	5' 0"	Sand	0.08 mm.[1]	40,000	Regular sewage	975	39	17	99.95	99.99	99.99
4	0.005 acre	5' 0'	River silt	0.04 mm.[1]	40,000	Regular sewage	195	42	33	99.99	99.99	99.99
5C	0.005 acre	5' 0"	Sand	0.22 mm.[1]	50,000	Regular sewage	5,600	3,200	2,440	99.73	99.29	99.33
6	0.005 acre	3' 8"	Sand[2]	0.35 mm.[1]	50,000	Regular sewage	2,160	760	590	99.90	99.83	99.84
9A	0.005 acre	5' 0"	Sand	0.17 mm.[1]	50,000	Regular sewage	1,290	384	252	99.94	99.91	99.93
10	0.005 acre	5' 0"	Sand[2]	0.35 mm.[1]	120,000	Regular sewage	2,565	470	400	99.88	99.90	99.89
Effluent, Contact Filters												
175	2.18 sq. ft.	5' 0"	Coke	¼" to 1"[3]	359,000	Effluent, strainer E.	553,300	74,100	60,100	36.70	47.90	48.10
376	10.37 sq. ft.	27 layers[4]	Slate	[3]	575,000	Regular sewage	1,105,600	342,400	290,300	47.20	23.70	20.30
377	6.70 sq. ft.	8 layers[5]	Slate	[3]	221,000	Effluent, Imhoff tank	2,123,500	184,700	129,800	.....	46.00	31.20

METHODS OF DISINFECTION

Bacteria tend to settle when present in quiescent sewage and some are carried down mechanically by the suspended matter, which contains large numbers of them. Consequently passing sewage through sedimentation tanks may result in the removal of a substantial percentage of the original bacteria, as shown in Table 171. The addition to the sewage of coagulants, such as lime, sulphate of alumina and sulphate of iron, causes a material increase in the efficiency of sedimentation and hence of bacterial removal. In practice, the number of bacteria remaining in the effluent after sedimentation or chemical treatment of sewage is large, and it frequently happens, especially in warm weather when bacterial life is very active, that such effluents contain even greater numbers of bacteria than the raw sewage. This is due to prolonged periods of sedimentation in which bacteria remaining in the supernatant liquid may increase in numbers and to the accumulations of sludge upon the bottom and sides of the tanks, from which large numbers of bacteria may be communicated to the sewage in spite of the initial removal of a substantial proportion of the organisms by sedimentation. These methods of treatment do not have bacterial efficiency sufficiently high to be of practical advantage under usual conditions, and are not generally considered to be methods of disinfection, although effecting some reduction in the numbers of bacteria.

Acids.—Most acids act as germicides under certain conditions, and considerable experimental work has been done to determine the quantities necessary for the disinfection of sewage. (Rideal, *Jour. Royal Sanitary Inst.*, vol. xxvi, page 392.)

The addition of 0.08 per cent. (6650 lb. per 1,000,000 gal.) of sulphuric acid to sewage has been found sufficient to destroy the germs of cholera and typhoid in 15 minutes, but the cost is prohibitive except in times of extreme emergency.

The sewage from cities containing industries from which spent acids are turned into sewers, as Birmingham, England, and Worcester, Mass., usually contains relatively small numbers of bacteria. The germicidal effect of such acids and acid salts may have an important bearing upon problems of sewage disposal.

Mine Drainage.—The disinfecting action of mine drainage, which often contains large quantities of acid and iron, may affect greatly the condition of streams receiving large quantities of sewage. A remarkable example is in the Monongahela and upper Ohio Rivers at and a short distance below Pittsburg, Pa. The city, having a population of 534,000 in 1910, is at the confluence of the Allegheny and Monongahela Rivers, the headwaters of the Ohio, and discharges its sewage untreated into these rivers at numerous points. The normal summer

flow of the Allegheny may be taken at 6000 cu. ft. and that of the Monongahela at 4000 cu. ft. per second. Both streams receive large quantities of mine drainage, but in the Allegheny the quantity is small in proportion to the flow of the river, while the quantity discharged into the Monongahela is sufficient to have a marked disinfecting action. The number of bacteria in the Allegheny water, which is generally slightly alkaline in spite of the mine drainage received, increases greatly between the city boundary line and its junction with the Monongahela, while the increase in the latter river is practically *nil*, although the proportion of sewage in it is much greater than in the Allegheny. The waters from these two rivers become thoroughly mixed a short distance below the city. The disinfecting action of the Monongahela upon the waters of the Allegheny is so great that in the Ohio River about 12 miles below Pittsburg the number of bacteria in the water is only slightly greater than in the water of the Allegheny immediately above the city, the water used after filtration for the municipal supply of Pittsburg.

The effect of different degrees of acidity and alkalinity upon the number of bacteria in the Monongahela River is shown by the following data furnished by Hazen and Whipple:

Acidity[1]	35	30	25	20	15	10	5	0	......
Alkalinity[1]	...	...	...	...	...	.....	.....	0	5
Bacteria[2]	150	280	360	500	750	1,400	2,800	7,500	30,000

[1] Parts per million.
[2] Numbers per cubic centimeter.

Heat.—It has been suggested that sewage might be sterilized by heat, at the same time distilling off ammonia which, it is claimed, would help pay the cost of the process. Thus far, however, this method has not had practical application and, under present conditions, it does not give promise of success. (Interim Report Royal Commission on Sewage Disposal, vol. ii, page 519.) Pathogenic bacteria can be killed by short exposure at temperatures above 60°C. Such procedure, found to give little promise of success abroad, would be even more dubious in America because of the great quantities of sewage produced per unit of population.

Ozone.—Many experiments have been made with ozone as a disinfectant for water, and a few plants, on a moderately large scale, have been installed. The use of ozone has been attended by two difficulties, however, first, the cost has thus far proved excessive and, second, ozone is so slightly soluble in water that it has been found very difficult to make its application efficient. While this process may perhaps be so developed as to be made practicable, it hardly seems likely that it will soon be put upon a sufficiently inexpensive basis to compete with calcium hypochlorite.

Copper Sulphate as Algicide.—This chemical has been used in numerous cases for killing algæ, for which it appears to be well adapted. It has also been used experimentally in a number of places for the disinfection of sewage and sewage effluents, but has been found to be less efficient for this purpose than chloride of lime and more costly.

While the growth of algæ in waters receiving sewage and effluents is probably generally advantageous, furnishing the plankton to absorb the products of decomposition and supply oxygen to prevent putrefaction, conditions may arise which make it desirable to prevent or destroy such growths.

At Pawtucket, R. I., in 1906, in order to free intermittent sand filters from a troublesome growth of Oscillaria, particularly persistent in the main distributors, all sewage flowing on to the beds was dosed as it left the settling tank, with copper sulphate, 1 part in 50,000 (167 lb. per 1,000,000 gal.), for a period of 15 days. At the end of this time the use of the copper solution was discontinued, and it was found that the nitrification of the beds had not been affected at all, while the Oscillaria had disappeared. (Report R. I. Board of Health, 1906, page 242.) Many experiments have been made to determine the efficiency of copper sulphate in destroying growths, and Moore and Kellerman have described some results in Bulletin 64, Bureau of Plant Industry, U. S. Department of Agriculture. They recommend thorough microscopic determination of the species of plants present in the water, that the quantity of copper sulphate applied may be in proper proportion to the quantity of water to destroy the organisms, for different species require different concentrations of copper sulphate. It is also necessary to know the chemical composition of the water, as it is occasionally necessary to apply lime or some other alkali with the copper to bring about the necessary chemical reactions. During the 8 years following the publication of that bulletin, the Department of Agriculture was frequently consulted concerning methods of killing algæ and fungi, and from this experience Kellerman prepared for the Eighth International Congress of Applied Chemistry a statement of the quantity of copper sulphate required to kill various species causing disagreeable odor or taste in water, which is summarized in Table 172. He also stated that the following amounts of copper sulphate, in parts per 1,000,000, should not be exceeded if it is desired to save the species of fish mentioned; Black bass, 2.10; carp, 0.3; catfish, 0.4; goldfish, 0.5; perch, 0.75; pickerel, 0.4; suckers, 0.3; sunfish, 1.2; trout, 0.14.

To apply copper sulphate, Moore and Kellerman advise placing the required quantity in a gunny-sack, attaching the bag to the stern of a boat and rowing slowly back and forth over the pond or sluggish stream, keeping the boat on each trip within 10 to 20 ft. of the previous path. One hundred pounds of the salt can be distributed in this way in an

hour. The number of boats should be increased as conditions require, as it is necessary to reduce the time consumed in applying the copper sulphate as much as possible.

TABLE 172.—COPPER SULPHATE REQUIRED TO KILL SPECIES OF ORGANISMS PRODUCING TASTES OR ODORS IN WATER (KELLERMAN)

(Parts per 1,000,000; 1 part per 1,000,000 = 8.33 lb. per 1,000,000 gal.)

Anabæna	0.09	Kirchneriella	5.00 to 10.00
Asterionella	0.10	Leptomitus	0.40
Beggiatoa	5.00	Microspora	0.40
Chara	0.20 to 5.00	Navicula	0.07
Cladophora	1.00	Oscillatoria	0.10 to 0.40
Cladothrix	0.20	Peridinium	2.00
Clathrocystis	0.10	Scenedesmus	5.00 to 10.00
Cœlosphærium	0.30	Spirogyra	0.05 to 0.30
Conferva	0.40 to 2.00	Ulothrix	0.20
Crenothrix	0.30	Uroglena	0.05
Euglena	1.00	Volvox	0.25
Fragillaria	0.25	Zygnema	0.70
Hydrodictyon	0.10		

Copper Sulphate as a Disinfectant.—A comprehensive study of disinfection of effluents of sewage treatment plants in Ohio was made in 1906–07 by Kellerman, Pratt and Kimberly. (Bulletin 115, Bureau of Plant Industry, U. S. Department of Agriculture.) Copper sulphate was used at four places. At a plant at a school, St. Mary's of the Springs, the application of 63 parts per 1,000,000 of copper sulphate to settled sewage reduced the bacterial count about 98.94 per cent. At Westerville, the effluent from a contact bed dosed with aerated septic tank effluent was treated at rates of 5 to 57 parts per 1,000,000. The results varied from 59 to 95 per cent. removal of the bacteria developing at 20°C., and were not much influenced by the proportion of disinfectant added. About 40 parts per 1,000,000 seemed to produce the best results, removing 93 per cent. of the bacteria developing at 20°C., 94 per cent. of the total colonies at 37°C. and 99 per cent. of the red colonies. More uniform results were obtained by disinfecting the effluent of intermittent sand filters at the Boys' Industrial School at Lancaster. The dose of sulphate ranged from 4 to 22 parts per 1,000,000 and the removal based on 20°C. was from 80 to 88 per cent. and on 37°C. from 64 to 97 per cent. of all bacteria and 85 to 100 per cent. of the red colonies were removed. The best results seemed to follow the use of 15 parts of sulphate per 1,000,000. The application of 5 to 116 parts per 1,000,000 of copper sulphate to intermittent sand filter effluent at Marion caused

irregular results, attributed to lack of time for the disinfectant to act. The period of contact[1] was 1 hour, about the same as at Westerville. The removal of bacteria developing at 20°C. was 58 to 92 per cent.; at 37°C. it was 53 to 93 per cent. for total colonies and 55 to 92 per cent. for red colonies, but the irregularity in the results was such that no rate of dosing was clearly the best.

Johnson and Copeland conducted experiments at the Columbus Testing Station in 1905 that showed a reduction of over 99 per cent. in bacteria by the application of 5 to 20 parts per 1,000,000 of copper sulphate. The time required, however, was 24 hours, a period prohibited in practice. Furthermore, they estimated the cost of copper sulphate at $5 to $10 per 1,000,000 gal. with doses of 10 to 20 parts per 1,000,000, a sum considerably exceeding the cost of sufficient calcium hypochlorite to accomplish equal results. The conclusions reached in the Ohio investigations were:

"Both calcium hypochlorite and copper sulphate have high germicidal values when acting upon partially purified sewage. Calcium hypochlorite is much more rapid in its action, is more nearly able to bring about complete disinfection at a lower cost and is less influenced by temperature and by the presence of carbonates." (Bulletin 115, Bureau of Plant Industry, U. S. Department of Agriculture, page 43.)

Chlorine and Its Compounds.—The chemicals coming under this head that have thus far proved of practical value as disinfectants are chlorine gas, liquid chlorine, calcium hypochlorite and sodium hypochlorite. Of these calcium hypochlorite, "bleach," has found the widest application, but liquid chlorine, put on the market after "bleach," is competing with it, being a more convenient form for use. Calcium hypochlorite is known also as "chloride of lime" and "bleaching powder." In some localities, where electric power is relatively inexpensive, sodium hypochlorite prepared electrolytically may prove less expensive and equally convenient.

Investigations at the Sanitary Research Laboratory of the Massachusetts Institute of Technology in 1906 showed that the effect of temperature upon the disinfection of trickling filter effluents was trifling with bleach but marked with copper sulphate, of which twice as much was required in winter as in summer. It was also found that most of the disinfecting action took place in the first 15 minutes after dosing. The disinfection of the suspended matter was as complete as that of the liquid. About 3.5 parts per 1,000,000 of available chlorine and a 1-hour contact period for the bleach to act reduced the total bacteria about 96 per cent.

[1] The time, after the disinfectant is added, during which a given quantity of sewage is retained in tanks or otherwise before being discharged into diluting water is called the period of contact and is important in affording the chemical opportunity to perform its work.

Investigations with crude sewage indicated that disinfection in this case was influenced by the degree of decomposition of the organic matter. Apparently the germicidal effect of the bleach was the same in all cases where an opportunity for the complete action was afforded, but the time of contact necessary for the full effect of the disinfectant to be developed varied with the character and amount of this organic matter. The experiments were made with 5 to 10 parts of available chlorine, averaging 7 or 8 parts, and contact periods of 0.5 to 4 hours. A large part of the chlorine was used up within 2 hours. (Water Supply Paper 229, U. S. Geological Survey.)

In 1909 experiments were made at Red Bank, N. J., by Phelps, with septic tank effluent. This was dosed with 11.5 parts of available chlorine, of which 5.5 parts were required to oxidize hydrogen sulphide and other easily oxidizable substances present, leaving 6 parts for germicidal work. The disinfected effluent remained 45 minutes in a settling tank and then an equal period in a second tank. The percentage of total bacteria removed was 99.7 in the first tank, leaving 2300 in the effluent, increasing to 99.8 in the second, leaving 1400 in the effluent, and the percentage removal of B. coli was 99.96 and 99.97 respectively, leaving 75 and 60 in the effluent.

Where the disinfection of septic effluent is necessary, Phelps suggested that it might be best under some circumstances to give the bleach an opportunity to act before the sewage enters the septic tank. If the quantity of disinfectant is so adjusted that none will pass into the septic tank, there will be a great increase in bacteria there, but not of pathogenic species. The development of the saprophytes in the tank, Phelps stated, would probably be of help in breaking down the organic matter in the water after the septic effluent has been discharged into it. (Water Supply Paper 229, U. S. Geological Survey, page 55.)

The disinfection of trickling filter effluents with bleach was investigated at Baltimore in 1908, using from 0.94 to 3.5 parts per 1,000,000 of available chlorine. The percentage reduction of bacteria at 20°C. was 96.6 and at 37°C., 94.9; of acid-formers at 37°C., 97, and of B. coli in Jackson's medium, 90. The numbers in the effluents were 4,300, 390, 70 and 200 respectively. The low removal of B. coli was attributed by Phelps to the growth of other bacteria than coli in the bile medium. (Water Supply Paper 229, page 60.)

Experiments were conducted at several Ohio plants in a similar way to those with copper sulphate, already described. At the Boys' Industrial School at Lancaster, the effluent from intermittent sand filters was dosed with 3.6 to 4.3 parts per 1,000,000 of available chlorine. The removal of acid colonies was complete. The removal of total bacteria at 20°C. was 99.8 and 99.9 per cent., and at 37°C. 99.6 and 99.7 per cent. There was an increase in dissolved oxygen, presumably due

to the liberation of oxygen from the bleach. At Marion, intermittent sand filter effluent was dosed with 1.5 to 3.8 parts of available chlorine. The reduction by the two doses named was 98.8 and 94.3 per cent. of the total bacteria at 20°C., 98.5 and 99.2 per cent. of the total at 37°C. and 100 and 99.9 per cent. of the acid-formers. Tests of contact-bed effluents at Marion, dosed with 2.9 to 5.0 parts of available chlorine, showed a reduction of 97.6 to 99.8 per cent. of total bacteria at 20°C., 99.4 and 99.5 per cent. of the total bacteria at 37°C., and 99.9 and 100 per cent. of the acid-formers. Runs were also made with septic tank effluent to which 4.3 to 7.6 parts per 1,000,000 of available chlorine were added, and others with doses of 7.3 to 48.5 parts. Good results were not obtained until the dose was raised to about 25 parts, but it proved impracticable to obtain conclusive results even then because the fluctuations in the large amount of suspended matter influenced the amount of disinfectant acting as a germicide. (Bulletin 115, Bureau of Plant Industry, U. S. Department of Agriculture.)

Experiments by Massachusetts State Board of Health.—A number of small-scale experiments by H. W. Clark and Stephen DeM. Gage are described in the report of the Massachusetts State Board of Health for 1911. Counts of total colonies on agar plates incubated 4 days at room temperature and total and red colonies on litmus lactose agar plates incubated 24 hours at body temperature, were made on all samples. Waters suitable for drinking in Massachusetts were usually found to contain less than 100 bacteria per cubic centimeter determined at room temperature, and the total bacteria developing on litmus lactose agar at body temperature is usually less than 10 per cubic centimeter and the red colonies usually less than 5 per cubic centimeter. This they called the "drinking water" or 100–10–5 standard. Two other standards containing respectively 10 and 100 times as many bacteria, called 1000–100–50 and 10,000–1000–500 standards, were also assumed to correspond approximately to the upper and lower limits of bacterial counts on river water receiving more or less pollution. The minimum, maximum and average quantities of disinfectant required to reduce the bacterial content of the several sewages and effluents to the specified standards are given in Table 173. The tabulated results are all based on a 2- to 4-hour period of contact.

The amounts of disinfectant by which reductions in the bacterial contents of 75, 90 and 99 per cent. were produced are given in Table 174. As the proportions of disinfectant in successive portions of the sample were increased uniformly, probably in many cases the amount stated as producing a given effect was somewhat greater than that necessary to produce the effect, but the next smaller amount tried was not sufficient to do so.

TABLE 173.—AMOUNT OF EFFECTIVE CHLORINE REQUIRED TO REDUCE BACTERIAL CONTENT TO PRESCRIBED STANDARDS

(Parts per 1,000,000; 1 part per 1,000,000 = 8.3 lb. per 1,000,000 gal. Abridged from 1911 Report of the Massachusetts State Board of Health, pages 345–350)

Sample	Sterile	100–10–5 Standard			1000–100–50 Standard			10,000–1,000–500 Standard		
		20°C. 100	40°C. Total 10	40°C. Red 5	20°C. 1000	40°C. Total 100	40°C. Red 50	20°C. 10,000	40°C. Total 1000	40°C. Red 500
Sewage:										
Maximum	+37.5	18.8	+37.5	+37.5	11.3	11.3	11.3	11.3	7.5	7.5
Minimum	26.3	3.8	7.5	3.8	−3.8	3.8	−3.8	−3.8	−3.8	−3.8
Average	34.7	9.4	22.5	21.3	6.6	7.5	7.6	6.6	4.7	4.7
Settled sewage:										
Maximum	+37.5	18.8	+37.5	+37.5	15.0	26.3	26.3	15.0	11.3	11.3
Minimum	+37.5	3.8	15.0	7.5	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Average	+37.5	9.0	29.3	21.0	8.3	15.0	11.3	7.5	6.8	6.8
Septic tank effluent:										
Maximum	+37.5	11.3	+37.5	+37.5	7.5	11.3	11.3	7.5	7.5	7.5
Minimum	+37.5	3.8	+37.5	+37.5	−3.8	7.5	3.8	−3.8	3.8	−3.8
Average	+37.5	8.8	+37.5	+37.5	6.3	8.8	7.5	6.3	6.3	6.3
Contact filter effluent:										
Maximum	+37.5	18.8	+37.5	+37.5	11.3	22.5	11.3	7.5	11.3	11.3
Minimum	37.5	3.8	30.0	11.3	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Average	+37.5	6.8	36.8	25.5	5.7	7.2	6.0	4.9	5.3	4.9
Trickling filter effluent:										
Maximum	+37.5	33.8	+37.5	33.8	7.5	18.8	7.5	3.8	7.5	7.5
Minimum	37.5	−3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	+37.5	6.9	26.9	20.4	4.4	5.7	4.1	3.8	4.1	4.1
Sand filter effluent:										
Maximum	+37.5	3.8	18.8	18.8	−3.8	3.8	3.8	[1]	−3.8	−3.8
Minimum	3.8	[1]	[1]	[1]	[1]	[1]	[1]	[1]	[1]	[1]
Average	17.2	3.8[2]	7.5[2]	8.5[2]	3.8[2]	3.8[2]	3.8[2]	[1]	−3.8[2]	−3.8[2]

− Less than stated amount. + More than stated amount.

[1] Initial number of bacteria less than standard.

[2] Based on figures given; some of samples being marked +.

The 10,000–1000–500 standard appeared fairly comparable with the 75 per cent. removal standard. The 99 per cent. removal standard, except with sand filter effluents, appeared materially lower than the 100–10–5 standard. With the very low initial bacterial content of the sand filter effluents, a 99 per cent. reduction in these numbers was a relatively severe requirement. In many cases the initial numbers of bacteria were lower in the sand filter effluents than required by the drinking-water standard.

TABLE 174.—AMOUNT OF EFFECTIVE CHLORINE REQUIRED TO PRODUCE STATED REDUCTIONS IN NUMBERS OF BACTERIA

(Parts per 1,000,000; 1 part per 1,000,000 = 8.3 lb. per 1,000,000 gal. Abridged from Report of the Massachusetts State Board of Health, 1911, pages 345–350)

Sample	99 per cent. reduction			90 per cent. reduction			75 per cent. reduction		
	20° C.	40°C.		20° C.	40°C.		20° C.	40°C.	
		Total	Red		Total	Red		Total	Red
Sewage:									
Maximum	11.3	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
Minimum	3.8	−3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	6.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
Settled sewage:									
Maximum	11.3	11.3	11.3	7.5	7.5	7.5	7.5	7.5	7.5
Minimum	3.8	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	6.8	5.3	6.8	4.5	4.5	4.5	4.5	4.5	4.5
Septic tank effluent:									
Maximum	7.5	7.5	7.5	−7.5	7.5	7.5	3.8	7.5	7.5
Minimum	−3.8	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	6.3	6.3	6.3	5.0	6.3	6.3	3.8	5.0	5.0
Contact filter effluent:									
Maximum	11.3	11.3	7.5	7.5	7.5	7.5	7.5	3.8	7.5
Minimum	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	5.3	4.9	4.5	4.5	4.2	4.2	4.5	3.8	4.2
Trickling filter effluent:									
Maximum	7.5	7.5	7.5	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Minimum	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	4.1	4.7	4.1	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Sand filter effluent:									
Maximum	37.5	11.3	7.5	11.3	7.5	3.8	−3.8	7.5	−3.8
Minimum	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	8.6	7.0	4.9	4.9	4.9	3.8	−3.8	4.9	−3.8

—Less than stated amount.

A general average of all the results indicated the effect of time of contact to be about as follows:

Hours of contact	1	2	4	6	24
Relative amounts of hypochlorite required	100	84	82	77	61

Experiments at Baltimore.—Experiments by Ezra B. Whitman, under the general direction of Calvin W. Hendrick, Chief Engineer of the Baltimore Sewerage Commission, with the collaboration of Phelps, indicated that considerably more calcium hypochlorite will be required to secure the same efficiency in disinfecting effluents from coarse, shallow trickling filters than in disinfecting those from a trickling filter 12 ft. deep, filled with broken stone varying from ½ to 1½ in. in diameter. The period of contact was in general 1½ hours. The results are stated in the annual report of the Commission for 1911, as follows:

"As a result of these disinfection experiments it was decided that an effluent could be obtained from the sprinkling filters at the main disposal plant which, by subsequent treatment with calcium hypochlorite, would compare favorably with the effluents which could be secured by treatment in supplementary sand filters. Although individual results were secured which showed a high percentage removal of bacteria with quantities of available chlorine varying from 1 to 2 parts per 1,000,000, yet these high percentage removals were not constant and the conclusions reached were that it would be necessary to use at least 3 parts of available chlorine per 1,000,000 gal. in treating the sprinkling filter effluents from the main disposal plant, should such treatment be deemed necessary."

Before making these experiments it had been assumed that to obtain an effluent of the "highest practical degree of purity," as required by the statutes, it would be necessary to subject the effluent from the trickling filters to secondary sedimentation and then pass it through intermittent sand filters. The plant required for such filtration was estimated to cost over $1,000,000, and the annual maintenance and operating expenses were estimated at $60,000 per year. The plant required for the disinfection of the trickling filter effluent was estimated to cost but a few thousand dollars, and the cost of maintenance and operation, using sufficient calcium hypochlorite to provide 3 parts per 1,000,000 of available chlorine, was estimated at $30,000 per year.

EXPERIENCE AT PROVIDENCE

For a number of years the sewage of Providence (population about 225,000) was treated with copperas and lime, passed slowly through large, shallow settling basins, and discharged at a depth of about 35 ft. into the Providence River, about 600 ft. off Fields Point, near the head of Narragansett Bay. The taking and marketing of shellfish from Narragansett Bay is rated third in value of the industries of Rhode Island. There was considerable agitation over the possible contamination of the shellfish beds, and the Rhode Island Commissioners of Shell Fisheries ruled that the marketed shellfish must conform in purity

to the standard of the United States Government, viz.; that three out of five specimens examined shall contain less than 1 B. coli in 0.1 cc. of shellfish liquor.

Experiments.—As a result of the movement to protect the consumer from contaminated shellfish, the city undertook in 1910 large-scale experiments to determine the efficiency of bleaching powder as a disinfectant of the sewage and effluent. The sewage was usually first given the regular treatment with about 750 lb. of lime per 1,000,000 gal., and then allowed to flow slowly through 3 settling tanks in parallel. The period of sedimentation was approximately 4 hours. The tank effluent was dosed with the solution of hypochlorite, after which it flowed slowly through 8 or 16 finishing tanks, the number being selected to give the desired period of contact, which could not be made less than 3½ hours. This period was taken as the length of time required to fill the basins with sewage at the rate it was flowing during the experiment. Little difference in the effect of the bleach was noticed whether 8 or 16 tanks were in use, but there was a marked difference due to variations in the quantity of disinfectant used. The contact period with 8 finishing tanks was 3.49 to 6.18 hours, and with 16 tanks it was 7.40 to 12.50 hours. The results of the tests are given in Table 175.

The increase in the total number of bacteria in the sewage passing slowly through the tanks after treatment with lime but not with bleach was from 1,700,000 to 12,700,000 during 11 hours, on one occasion, and during 19 hours, on two occasions, it was from 1,200,000 and 1,300,000 to 14,500,000 and 22,300,000 respectively.

The use of bleach alone, without previous treatment with lime, did not prove as efficient as the application of bleach to the sewage after it had been treated with lime and had passed through the roughing tanks. Experiments 8, 9, 18 and 19 (Table 175) illustrate this. The quantity used in experiment 9, 4.15 parts, proved entirely inadequate, while the same amount in experiment 8 did fairly good work. The latter sewage was the weaker, being made up in part of the night flow, which probably accounts for the difference in the results of the two experiments.

Less than 4 parts of chlorine gave erratic results, but there was a remarkable uniformity in efficiency with from 4 to 6 parts, with respect to bacteria developing at room and body temperatures and acid colonies.

The effect of a prolonged period of contact was to increase greatly the number of bacteria in the final effluent, those developing at body temperature as well as room temperature.

An important result of these tests was the demonstration that by the addition of bleach the period of stability of a chemical effluent can be greatly increased, as shown by methylene blue tests. Three samples

of untreated chemical effluent proved to be putrescent within 1 day. With chlorine below 3.50 parts per 1,000,000 the samples were stable from 1½ to 7½ days, but with 3.50 parts or more of available chlorine, the samples were uniformly stable for 20 days. All tests were made during August and September. The stability tests were all made upon small samples incubated in closed bottles. Were the samples moderately diluted with a contaminated river or pond water, bacterial development would have been much more rapid and putrefaction might have set in much earlier than when the effluent was tested by itself.

TABLE 175.—RESULTS OF EXPERIMENTS UPON DISINFECTING ACTION OF BLEACHING POWDER UPON CHEMICAL EFFLUENT, PROVIDENCE

Finishing tanks used	Lime, pounds per mil. gal.	Parts available chlorine per mil.	Bacteria per cc. in final effluent		Bacteria per cc. in final effluent		Bacteria per cc. in final effluent	
			Total at 20°C.	Per cent. removed	Total at 37°C.	Per cent. removed	Red col's. at 37°C.	Per cent. removed
8	0	2.00	....	....	200	99.97	−100	99.00
8	0	2.00	....	....	875,000	increase	−100	90.±
8	705	2.60	95,000	90.00	....	....	....	....
8	869	3.00	90,000	91.81	....	....	7,100	79.71
8	774	3.25	3,600	98.97	....	....	400	96.00
8	755	3.50	30,500	86.45	....	....	2,050	90.24
8	800	4.00	8,000	96.67	....	....	1,000	90.00
8	0	4.15	....	....	100	99.95	−100	90.±
8	0	4.15	....	....	2,500,000	increase	3,100	increase
8	765	4.60	225	99.55	....	....	50	96.67
8	731	4.60	1,800	99.28	....	....	120	99.56
8	886	4.69	....	....	−100	99.94	−100	99.87
8	886	4.69	....	....	100	99.91	100	99.75
8	738	5.40	750	99.70	....	....	525	89.50
8	589	5.80	4,750	99.50	....	....	100	99.70
8	721	5.89	....	....	300	99.40	100	99.60
8	721	5.89	....	....	100	99.94	−100	99.86
8	0	7.38	....	....	125,000	83.33	100	99.80
8	0	7.38	....	....	90,000	83.08	100	98.50
16	703	3.00	29,000	97.36	....	....	400	98.86
16	724	3.25	14,500	96.78	....	....	2,300	84.67
16	723	3.87	830,000	increase	916,700	5.5	....	[1]
16	820	3.96	....	....	133,600	19.62	....	[1]
16	742	4.25	300	99.70	....	....	−100	99.00
16	636	4.41	....	....	458,000	increase	....	[1]
16	777	4.57	....	....	8,400	98.27	....	[1]
16	828	4.64	100	99.95	13,500	95.45	....	....
16	844	4.70	1,500	99.40	....	....	....	....
16	853	4.79	700	99.00	10,600	96.72	....	....
16	147	4.95	....	....	8,800	98.74	....	....
16	746	6.56	....	....	17,900	96.54	....	....

[1] On these 4 days bleaching powder added *only* while pumps were running (12–14 hours).

— Means less than.

The character of the sewage and effluent is shown by the average free and albuminoid ammonia for August and September, given in Table 176.

TABLE 176.—ANALYSES OF SEWAGE AND EFFLUENT AT PROVIDENCE ON DAYS OF EXPERIMENTS IN AUGUST AND SEPTEMBER, 1910

(Parts per 1,000,000)

	Free ammonia	Albuminoid ammonia		
		Total	Dissolved	Suspended
Sewage, average of 15 samples.	19.60	9.93	4.79	5.14
Effluent, average of 15 samples.	21.90	5.45	4.68	0.77
Percentage reduced		45.1	2.3	85.0

During the earlier experiments bleach was added to the sewage only while the pumps were in operation, from 12 to 15 hours per day, with the expectation that the small night flow between pumping periods would be disinfected in passing through the tanks. It was found, however, that the untreated portions were contaminating the whole flow, so that it was necessary to apply the bleach continuously throughout the 24 hours.

Disinfection Practice.—The results of the experiments proved so satisfactory that disinfection was adopted as a part of the regular treatment. During the first 5 months in 1911, the sewage was treated substantially as during the experiments, Table 177. The results of treatment without lime during 1912 and 1913 are given in Table 178. In 1914 the use of lime was resumed. In August of that year about 14,000 lb. were added daily at the same time that the bleach was added. The cost of sedimentation and disinfection during 1912 was $2.85 per 1,000,000 gal. of sewage treated, and $2.50 during 1913, according to the annual reports of the City Engineer.

APPARENT SELECTIVE ACTION OF OXIDIZING DISINFECTANTS

Experiments at Boston and Providence indicate that the action of hypochlorites upon sewage and effluents causes a greater percentage reduction of bacteria growing at body temperature and acid-formers than of ordinary bacteria growing at room temperature. These results have led some investigators to conclude that hypochlorites have a selective action most intense on organisms of fecal origin and that, therefore, the reduction of danger, resulting from disinfection, is even greater than indicated by the reduction in numbers of bacteria growing at room temperature. On the other hand, Clark and Gage found

TABLE 177.—RESULTS OF LIME TREATMENT AND DISINFECTION OF CHEMICAL EFFLUENT AT PROVIDENCE, JANUARY–MAY, 1911

Month, 1911	Available chlorine, parts per million	Lime, lb. per million gal.	Bacteria per cc. (total count at 37°C.)					Acid formers per cc.				
			Sewage	After lime treatment	Final	Per cent. removed		Sewage	After lime treatment	Final	Per cent. removed	
						Lime	Whole treatment				Lime	Whole treatment
January	3.6	417	69,800	30,000	1,340	57.02	98.02	36,100	17,600	670	51.25	98.14
February	3.6	419	88,000	14,300	3,190	83.75	96.38	43,300	10,400	1,550	75.98	96.42
March	3.7	466	63,700	32,300	430	49.29	99.33	35,600	18,500	200	48.03	99.44
April	4.1	444	43,900	26,500	200	39.63	99.54	38,800	19,600	120	49.49	99.69
May	5.0	492	83,900	245,000	200	−192.01	99.77	43,200	138,000	140	−319.44	99.68

TABLE 178.—OPERATING RESULTS OF DISINFECTION PROCESS AT PROVIDENCE, 1912 AND 1913

Month	Available chlorine, parts per million		Total bacteria removed, on agar at 37°C.; percentage		B. coli removed, lactose bile at 37° C.; percentage		Month	Available chlorine, parts per million		Total bacteria removed, on agar at 37°C.; percentage		B. coli removed, lactose bile at 37° C.; percentage	
	1912	1913	1912	1913	1912	1913		1912	1913	1912	1913	1912	1913
January	5.34	4.86	98.2	75.7	99.1	98.2	July	7.70	5.43	70.6	70.8	95.1	99.2
February	5.36	4.07	96.8	74.9	99.4	98.4	August	8.25	4.68	59.9	66.9	98.8	97.1
March	3.41	4.88	92.4	72.9	97.1	97.6	September	8.46	3.46	70.2	59.3	96.2	96.7
April	3.90	5.35	99.2	76.9	94.3	99.6	October	7.41	3.59	66.7	67.1	99.1	94.6
May	5.26	5.46	99.9	76.8	97.3	97.5	November	5.23	3.74	71.7	55.1	98.3	94.7
June	7.07	5.44	93.3	67.1	98.0	97.9	December	4.25	3.54	71.3	69.3	98.5	96.9

that the quantity of disinfectant required to reduce the bacteria growing at body temperatures and acid formers to definite standards was greater than that required to reduce correspondingly the bacteria growing at room temperature. As they adopted in many experiments more severe standards than those of other experimenters, the difference in conclusions may be attributed to certain resistant forms which had to be removed in order to reduce the number of bacteria to the "drinking water" standard, but were not removed in the other experiments. In fact, Clark's experiments based on percentage removal do not indicate that a greater quantity of hypochlorite is required to reduce the body temperature counts by a definite percentage than the room temperature counts.

According to Clark, the room temperature counts include many types of bacteria whose numbers are subject to fluctuations without sanitary significance. The body temperature counts, on the other hand, while much smaller than the room temperature counts, show more closely the numbers of bacteria present of types of fecal origin. In natural waters and sewages there is ordinarily an approximately definite ratio between the counts obtained at the two temperatures. Higher counts at body temperature than at room temperature are relatively rare. In both waters and sewages treated with oxidizing disinfectants relatively higher counts at body temperature than at room temperature in the same sample are frequent. The disinfectants were found by Clark to be slow in destroying certain types of bacteria which show up on the body temperature plates but fail to develop on room temperature plates. These resistant types are not of the usual colon type, Clark held, because of their small proportion of red colonies on the body temperature plates, nor are they, in the majority of instances, spore-forming bacteria, although a small proportion of spore-formers are found among them.

INCREASE IN BACTERIA AFTER DISINFECTION WITH HYPOCHLORITE

The increase in bacteria after disinfection with hypochlorites has been observed by many investigators. Phelps reported in Water Supply Paper 229, page 46, that pathogenic bacteria did not increase in this way outside the body. Clark and Gage found in 1911 that while the bacteria in a number of experiments had been reduced to very small numbers after 2 to 4 hours, after 24 hours a very large secondary bacterial increase had taken place. The relative occurrence of these after-growths in sewage and effluents and the largest amount of disinfectant in each experiment which failed to prevent such growths are shown in Table 179.

TABLE 179.—RELATIVE OCCURRENCE OF BACTERIAL AFTER-GROWTHS IN DISINFECTED SEWAGE AND EFFLUENTS AND THE LARGEST QUANTITY OF DISINFECTANT IN EACH EXPERIMENT WHICH FAILED TO PREVENT SUCH GROWTHS

(Report Massachusetts State Board of Health, 1911, page 356)

	Number of experiments made	Number of experiments with increase in bacteria			Largest amount hypochlorite in each experiment which failed to prevent bacterial increase (parts per 1,000,000)		
		20° C.	40°C. Total	40°C. Red	20°C.	40°C. Total	40°C. Red
Raw sewage......	4	1	1	2	7.5	3.8	3.8, 7.5
Clarified sewage..	10	5	5	7	3.8	3.8(4)	3.8(6)
					7.5(3)	....	....
					15.0	11.3	11.3
Contact filter effluent.	10	2	2	4	15.0	3.8	3.8(2)
					18.8	7.5	7.5(2)
Trickling filter effluent.	12	3	2	2	7.5(2)	3.8	3.8
					11.3	7.5	11.3
Sand filter effluent.	7	0	0	0	....	....	....

Note.—Figures in parentheses indicate number of experiments in which identical quantities were required.

Clark and Gage explain the freedom of sand filter effluents from after-growths as due to the dissolved oxygen in them. When once the oxygen liberated by hypochlorites was all absorbed, the disinfecting value of the hypochlorites was exhausted. Sewages and effluents from contact beds and trickling filters contain substances having some affinity for this oxygen, so that only a part of it was available for destroying the bacteria. If all the oxygen was absorbed before complete disinfection occurred, the remaining bacteria could multiply without restraint. In fact, by eliminating a large proportion of the bacteria, including probably all of certain species, the conditions became peculiarly favorable for rapid multiplication of the remaining bacteria since bacterial equilibrium had been destroyed and natural bacterial antagonism had been more or less eliminated; certain organic matters having been oxidized, the dissolved matters may perhaps have become better suited for bacterial food than previously. In every case included in Table 179 practically all bacteria were destroyed, but a few were left which found themselves in a peculiarly favorable environment after the disinfectant was exhausted. That they multiplied under these conditions to 100 to 1000 times their original numbers is in accord with the usual growth of bacteria under favorable conditions. But

the sand filter effluents were all highly oxidized and contained little organic matter to absorb the oxygen from the disinfectant, so that the smallest amount of hypochlorites used in the experiments made with them, 3.8 parts per 1,000,000, was sufficient not only to destroy practically all the bacteria, but to leave some traces of disinfectant still in solution by which the germinating bacteria were destroyed and after-growths prevented.

COMPARATIVE GERMICIDAL EFFICIENCIES OF CHLORINE AND CHLORINE COMPOUNDS

Studies by Phelps to determine the comparative germicidal efficiencies of chlorine and some of its compounds on trickling filter effluents

TABLE 180.—RELATIVE GERMICIDAL PROPERTIES OF CHLORINE AND SOME OF ITS COMPOUNDS. ALL NUMBERS CONVERTED TO A UNIFORM BASIS OF 1,000,000 INITIAL BACTERIA PER CUBIC CENTIMETER

(Water Supply Paper 229, U. S. Geological Survey, page 60)

Series.	Source of chlorine	Available chlorine (in parts per mil.)	Total number of remaining bacteria per cubic centimeter		
			At end of 30 minutes	At end of 1 hour	At end of 2 hours
I	Free chlorine	3	650	390	280
	Sodium hypochlorite	3	500	270	230
	Potassium hypochlorite	3	410	260	280
	Potassium chlorate	3	800,000	900,000	1,000,000
	Potassium perchlorate	3	750,000	1,400,000	1,800,000
II	Free chlorine	2	17,000	13,000	17,000
	Sodium hypochlorite (a)	2	15,000	6,000	6,000
	(b)	2	4,600	2,100	3,400
III	Free chlorine	2	19,000	18,000	23,000
	Sodium hydroxide and chlorine (c)	2	23,000	14,000	19,000
	(d)	2	22,000	12,000	18,000
	(e)	2	23,000	9,000	10,000
	(f)	2	19,000	7,500	8,000

(a) Electrolytic.
(b) From bleaching powder.
(c) Chlorine added 30 minutes after the hydroxide.
(d) Chlorine added 20 minutes after the hydroxide.
(e) Chlorine added 10 minutes after the hydroxide.
(f) Chlorine added with hydroxide.

gave the results summarized in Table 180. Phelps drew the following conclusions from the experiments:

"The results indicate plainly that hypochlorites are the most efficient germicides. Gaseous chlorine is almost as good, but in each series the free chlorine is somewhat inferior to the hypochlorite. Chlorates and perchlorates have almost no value in disinfection. The formation of these compounds in the electrolytic cell is, therefore, a total waste of energy, and should be prevented as far as possible. Production of these compounds explains in large measure the inefficiency of hypochlorite cells. Hypochlorites made electrolytically are slightly inferior to the market product, but this difference would probably be inappreciable in large-scale tests, where the conditions under which the hypochlorites are prepared are more nearly those of commercial practice. Hypochlorites of different bases evidently have the same value."

EFFECT OF CALCIUM HYPOCHLORITE ON COLON AND TYPHUS BACILLUS

To determine the relative effect of disinfection with hypochlorites upon the colon and the typhus bacillus, Phelps introduced into emulsions of the two organisms in tap water different quantities of hypochlorite solution, the available chlorine ranging from 3.5 to 6 parts per 1,000,000 and averaging 5 parts. The results of the individual tests varied greatly because, Phelps concluded, of differences in the character of the growths and the amounts of organic matter introduced with the organisms. Twelve sets of tests were made and Phelps believed that the average results given in Table 181 showed the true comparative resistance of the two organisms to the disinfectant. These indicate, as have other considerations already noted, that B. coli may reasonably be regarded

TABLE 181.—COMPARATIVE RESISTANCE TO CALCIUM HYPOCHLORITE[1] OF B. TYPHI AND B. COLI IN AQUEOUS EMULSION

(Water Supply Bulletin 229, U. S. Geological Survey)

Tests made at end of	Removal of bacteria (per cent.)	
	B. typhi	B. coli
20 minutes	90.5	92.0
40 minutes	98.2	98.0
1 hour	99.45	99.53
2 hours	99.60	99.70
4 hours	99.92	99.96
18 hours	99.99	99.99

[1]Average available chlorine, 5.0 parts per 1,000,000.

as test organisms in disinfection work and that the process may be expected to destroy typhoid organisms present at least as thoroughly.

DISINFECTION OF SEWAGE BEFORE FILTRATION

The Massachusetts State Board of Health (report, 1908, page 362) has made a few experiments to determine the effect of bleaching powder added to the raw sewage upon purification by filtration. Sewage treated with 25 parts per 1,000,000 of available chlorine was applied for 3 months to a sand filter which had been producing a well nitrified effluent, without causing any decrease in the nitrification.

Similar experiments were tried with an experimental trickling filter, the disinfectant being first applied at the rate of 5.0 parts available chlorine and gradually increased to 50 parts. The latter rate was maintained for 3 months, during which the nitrates decreased from 20 to 1.5 parts per 1,000,000, but nitrification did not cease.

Dr. W. P. Dunbar found that the addition of chloride of lime to sewage before filtration reduced bacterial activity in the upper layers of the filter, but that in the lower layers even the sensitive nitrifying organisms remained undisturbed, and the processes of purification and oxidation continued unhindered. ("Principles of Sewage Treatment," page 241.)

The addition of disinfectants to sewage before it is applied to filters may be desirable during epidemics where the sewage is known to be seriously infected. It may also be valuable in preventing the diffusion of objectionable odors from the spraying of sewage when dosing trickling filters, as demonstrated by John D. Watson at Birmingham, England (Report Metropolitan Sewerage Commission of New York, 1914, page 188), in reducing organic growths tending to clog such filters and in killing the little moth flies which thrive in and about the filters. Its effectiveness for this purpose has been demonstrated by Harry J. Hanmer, City Engineer, Gloversville, N. Y., who has used it with some success, on the advice of the authors. Twelve pounds of dry powder was added to each of three 16,720-gal. doses (equivalent to 28.7 parts per 1,000,000) on one day, and the same treatment was repeated on the second day thereafter. The rate of application was approximately 12 lb. per acre per day, equivalent to about 12 lb. per 1,000,000 gal. per day. This treatment killed the larvæ and young flies but did not appear to kill the full-grown insects. It materially reduced the fly nuisance about the filters.

CHEMISTRY OF DISINFECTION WITH BLEACHING POWDER

Oxidizing agents like calcium hypochlorite, permanganates and ozone have long been recognized as germicides. They apparently release in

a nascent state a portion or the whole of their oxygen, which instantly seeks its affinity in the most susceptible substances at hand, among which are the bacteria.

Chlorine and several of its compounds, such as calcium, sodium and potassium hypochlorite, because of their power of directly or indirectly liberating oxygen and their relative cheapness, have been extensively used as oxidizing and disinfecting agents. Whichever of these substances is used, the reactions taking place result in the production of hypochlorous acid, $HClO$. This is very unstable and is at once dissociated into hydrochloric acid (HCl) and oxygen (O). The former combines with such alkaline substances as may be available, forming an inert salt, such as calcium chloride, $CaCl_2$, while the latter attacks the sensitive organic substances.

Each molecule of hypochlorous acid contains one atom of chlorine and one of oxygen, and as an atom of oxygen has twice the combining power of the chlorine atom, it follows that the oxidizing power of the compound is twice as great as its chlorine content. Thus, if a given sample of bleaching powder actually contains 20 per cent. of chlorine in the form of calcium hypochlorite, its potency is double this and therefore it is said to contain 40 per cent. "available chlorine," though in reality it does not. The words "available chlorine" simply constitute a term by which is expressed the power of the compound to oxidize or disinfect. Table 182 gives the equivalents of parts per million of available chlorine in pounds of bleach of average quality per 1,000,000 gal. of sewage.

TABLE 182.—PARTS AVAILABLE CHLORINE, EQUIVALENT POUNDS CALCIUM HYPOCHLORITE AND COST OF DISINFECTANT PER MILLION GALLONS SEWAGE

(Calcium hypochlorite assumed to contain 33⅓ per cent. available chlorine)

Parts available chlorine per 1,000,000	Pounds calcium hypochlorite per 1,000,000 gal.	Cost of disinfectant per 1,000,000 gal. sewage		
		1.25 cts. per lb.	2 cts. per lb.	5 cts. per lb.
0.5	12.5	$0.156	$0.25	$0.625
1.0	25.0	0.313	0.50	1.25
5.0	125.0	1.56	2.50	6.25
10.0	250.0	3.125	5.00	12.50
25.0	625.0	7.815	12.50	31.25

Bleaching powder may be a compound represented by the formula $CaOCl_2$, or it may be a mixture of $CaOCl_2$ and $CaCl_2$. Chemists do not agree upon this point.[1] It makes little difference which is correct,

[1] Albert H. Hooker, in his book on "Chloride of Lime in Sanitation," suggests that $4CaOCl_2,2Ca(OH)_2,5H_2O$ represents the constitution of commercial bleach.

because when once the bleach is in solution there results a mixture of dissolved calcium chloride, $CaCl_2$, and calcium hypochlorite, CaO_2Cl_2. The latter is the active oxidizing or disinfecting agent. The CaO_2Cl_2 is decomposed by the carbonic acid in the water into water and hypochlorous acid, the latter being broken up immediately into hydrochloric acid and oxygen.

According to Prof. Charles Gilman Hyde the following typical probable composition of a high grade of bleaching powder has been calculated by Prof. H. B. Cornwall from the terms of an actual analysis:

Calcium oxychloride ($CaOCl_2$)	=	64.93 per cent.
Calcium chloride ($CaCl_2$)	=	1.28 per cent.
Calcium chlorate ($Ca(ClO_3)_2$)	=	0.34 per cent.
Calcium hydroxide ($Ca(OH)_2$)	=	19.64 per cent.
Calcium carbonate ($CaCO_3$)	=	1.51 per cent.
Calcium sulphate ($CaSO_4$)	=	0.25 per cent.
Oxides of sodium, potassium, magnesium, aluminum, iron and silicon	=	2.52 per cent.
Moisture	=	9.95 per cent.
Total		100.42 per cent.

MANUFACTURE OF AND MARKET FOR BLEACHING POWDER[1]

Bleaching powder, practically considered, is known in the trade as chloride of lime, and is a white product having a faint odor of hypochlorous acid. It was first manufactured by Messrs. Tennant & Co. in Glasgow, in 1799 and was a sequel to the manufacture of soda ash by the Leblanc process immediately antedating it. Each has a common parentage in common salt, sodium chloride.

Bleaching powder has been produced on a large scale for over a century and is classed as one of the staple heavy chemicals. It is very largely used by bleacheries, paper mills, the cotton oil industry and numerous other smaller consumers. It is packed in wooden casks of about half a gross ton or more, and recently in sheet-iron drums of about one-fourth gross ton.

Although the consumption of bleaching powder has greatly increased in recent years the production has been on an even larger scale. As a result, the price sank to about $25 per ton in 1912 and manufacturers resorted to making other products to dispose of their surplus chlorine gas. There is no prospect that the market price of bleach will increase on account of its use for water and sewage disinfection.

Bleaching powder has been manufactured on a large scale by chemical processes for over a century past, but during recent years electrochemical

[1] By courtesy of Martin L. Griffin.

processes have invaded the field to a considerable extent, particularly where power is cheap. It is made from common salt.

By the chemical process salt is first treated with sulphuric acid, producing sodium sulphate, commonly known as salt cake, and muriatic acid. It is this acid from which chlorine gas is obtained, and the next step in the process consists in mixing the acid with black oxide of manganese and heating the mixture, when chlorine gas is given off. The manganese oxide is recovered for use over again by what is known as the Weldon process. The chlorine gas is led off into large chambers containing slaked lime placed in ridges, which absorbs it, producing the final product. It is then packed and ready for shipment. This process was so well conceived almost from the beginning that it has stood the test of time with no essential modification, and it is doubtful if it will be rivalled by any other than electrochemical processes.

While the knowledge that the elements possess electropositive and negative properties is as old as the discovery of the elements themselves, no use, of any account, was made of this knowledge leading up to the manufacture of bleach until near the close of the last century. With the great development in the science of electrical engineering came the search for a useful field for its products and attention was soon directed to the electrolysis of salt. The first attempt to make bleaching solutions in this way consisted in passing a low voltage current through a solution of salt, resulting in a dissociation of some of the salt into its components, sodium and chlorine, which react in the bath to make sodium hypochlorite. This is a convenient method of making a weak solution of hypochlorite and has been proposed as a convenient and practical method of disinfecting garbage and sewage in our cities, especially where sea water is available. The method is employed by many small concerns, such as laundries and knitting mills, and where convenience takes precedence over economy. When, however, it is proposed to make bleach on a large scale efficiency of process in every detail must be studied. There is a great variety of decomposing cells, but all employ some means of keeping the components of the salt separate from each other, and freeing them from the mother brine. The chlorine is led off into absorption chambers to be taken up by the slaked lime, if bleaching powder is to be made, or led into lime water if bleaching solution is to be the final product.

MANIPULATION IN DISINFECTING SEWAGE WITH BLEACHING POWDER

The price paid for bleaching powder and the size of the package in which it is received will depend largely upon the quantity required. In large plants it will be purchased in carload lots and will be delivered

in wooden casks or sheet-iron drums, and can be stored for considerable periods of time without appreciable deterioration. If the quantities to be used are small they can be purchased of dealers in chemical supplies in small sheet-iron drums, in tin boxes or in bulk. If large drums are purchased and the quantity to be used is small, requiring the storage of the powder for a considerable time after the drum is opened, or if purchased in bulk, there will be a depreciation in quality due to decomposition and consequent loss of hypochlorite.

In practice the disinfectant is best introduced into the sewage as a solution, the quantity of bleach applied from hour to hour being varied by increasing or decreasing the flow of disinfecting solution in accordance with the volume or quality of the sewage to be treated. The direct introduction of the dry powder, as is now successfully accomplished with sulphate of alumina in water purification, has been suggested but thus far has not been attempted and does not seem likely to prove practicable on account of the hygroscopic nature of bleach, due to small quantities of calcium chloride contained in it.

As the dust and fumes from bleaching powder readily attack the membranes of the throat and nose, appliances should be provided to prevent, as far as possible, the escape of fumes and dust. This can be accomplished by carefully opening and handling the packages and by emptying them into the water as soon as they are open. In some cases it may be desirable to place the package, immediately after it is opened, under the surface of the water and empty the bleaching powder without its coming into contact with the air.

In most cases it will be desirable to provide a dissolving tank and a diluting or solution tank. The dissolving tank should be of sufficient capacity to provide for a 3 per cent. solution, or say 4 gal. of water for every pound of bleaching powder to be dissolved at one operation. Where mixing is done by hand fairly strong solutions require somewhat less labor and involve fewer difficulties in mixing than the more dilute solutions. The water should be allowed to stand several hours in contact with the bleaching powder, preferably from 12 to 24 hours, before diluting, to permit the lumps of bleaching powder to become soft. The fine material dissolves readily, but the lumps must be broken up or the full value of the bleaching powder will not be utilized.

After the bleach has been dissolved, the solution should be run into a larger tank and diluted sufficiently to make a solution of a strength of from 1 to 2 per cent. This solution should be sufficiently stirred to cause a uniform mixture of the stronger solution with the diluting water. Solutions of this strength are more convenient for use than stronger solutions, because a greater quantity of the liquid can be applied to a given quantity of sewage, making good mixing of the two an easier matter, and also because a slight adjustment of the appara-

tus for measuring the bleaching liquid will not make such a great change in the quantity of disinfectant applied.

At the present time there seems to be no reliable quick test to determine the quantity of bleaching powder required to disinfect a given sewage or reduce its bacterial content to any specified standard. Rideal, in his "Sewage and the Bacterial Purification of Sewage" (third edition, page 186), has suggested that the amount of chlorine necessary for disinfection may be approximately estimated from the oxygen consumed by the permanganate test, 5 minutes. Clark and Gage found that in about one-half the samples tested, this method gave results agreeing fairly well with those obtained by their experiments; in about one-quarter of the samples the amount of disinfectant computed from the test was considerably less than the amount which the experiments showed to be necessary to produce any practical effect; in the remainder of the samples the amount computed from the oxygen consumed was several times in excess of that required to produce complete disinfection. They concluded that, while the oxygen-consumed values may furnish some idea of the quantity of disinfectant to be used, the possibility of error will be so large that little reliance can be placed on this method. Substantially the same conclusion was reached by Kellerman, Pratt and Kimberly.

Dunbar has suggested ("Sewage Treatment," page 238) that information regarding the quantity of hypochlorite required might be obtained by determining the quantity of free chlorine present in the sewage after disinfection was practically completed. Analyses made then are, of course, confirmatory of the operator's previous judgment and can be of use only in aiding his judgment in the future. Such tests may be of considerable assistance, but they should be accompanied at first, if not continuously, by bacterial counts and other chemical examinations of the raw and treated sewage. The quantity of free chlorine present depends not only upon the quantity of hypochlorite applied and the chemical composition of the sewage but also and largely upon the period elapsing between the dosing and the test, during which the hypochlorite has had time to attack and be used up by the organic matter. Under ordinary working conditions, it is practicable to select a definite period of contact after which the test should be made, and it is to be expected that a definite quantity of free chlorine present at the end of such period will be a reasonably accurate indication of the effect of the disinfectant upon the bacteria of the sewage. The period selected should be of suitable length to cause a substantial reduction and assure a material residual supply of the chlorine. It need not be coincident with or as long as the period of contact allowed for the action of the hypochlorite upon the sewage under working conditions, which should be sufficiently long to allow

the utilization of all disinfectant added to the sewage. This actual time required will vary with the temperature and dilution of the sewage, and the period should be based upon the maximum requirements for the conditions at hand.

In practice the method of determining the quantity of disinfectant to be added will depend very largely upon local conditions. In cities where the sewage contains large quantities of industrial wastes affecting its bacterial content injuriously, there are likely to be times when there will be few bacteria present. Such conditions, however, depend upon discharge of industrial wastes which probably vary from day to day in quantity and in strength, and it will only rarely be possible to vary the dose of disinfectant added to take advantage of these conditions. In view of the varying quality of sewage, it seems necessary to test the efficiency of disinfection frequently and to regulate the quantity of applied disinfectant by the tests on previous days. With such a method of regulation it will always be necessary to apply an excess of disinfectant, if the operator wishes to be sure of accomplishing the desired result, because of the danger of the sewage requiring somewhat more disinfectant than did that tested on the day previous. If it is found that the sewage, at the time of maximum strength, requires 10 parts of available chlorine to accomplish a certain desired result, the quantity may be reduced at the time of minimum strength to perhaps 5 parts, as determined by tests made upon similar sewage on previous days. It will also be found that the quantity required to treat the sewage on a Sunday or a holiday, will vary from that required on other days. Such variations will require the use of good judgment by the operator and cannot be predicted with exactness by chemical tests.

Providence Practice.—A dissolving tank and equipment designed by Bugbee, put into operation at Providence about July 1, 1912, has proved very satisfactory. It is cylindrical with a conical hopper bottom. It is built of brick and covered by the floor above, which is placed directly on top of the tank, with a tight joint between the two, as shown in Fig. 209. From the extreme bottom a suction pipe leads to a 3-in. horizontal centrifugal pump driven by a 5-h.p. electric motor. This pump has a cast-iron shell and brass shaft and runner. The discharge pipe connects tangentially with the mixing tank at a point near the surface of the solution and with the dosing tanks. By closing the valve leading to the dosing tank and opening the valve leading back to the mixing tank it is possible to pump the solution over and over through the tank, thus thoroughly mixing the bleaching powder with the water. When the mixing has been completed, the valve on the pipe leading to the mixing tank is closed and the valve on the pipe leading to the dosing tank is opened, after which the solution is pumped

to the dosing tank. There is an opening 18 in. in diameter, in the floor, directly over the center of the mixing tank. Pointing upward into the center of this opening is a 2-in. wrought-iron water pipe.

The bleaching powder is furnished in metal drums holding about 800 lb. each. When it is desired to make up a tank of bleaching powder solution, a hole about 8 in. in diameter is cut in the bottom of a drum, thus avoiding the escape of chlorine gas which always collects at the top end. The drum is then inverted over the hole in the floor and so placed that the 2-in. water pipe points into the center of the opening in the drum. When the drum is in proper position the water is turned on and the bleaching powder washed out into the tank below, without the escape of any dust into the room and consequent discomfort to the

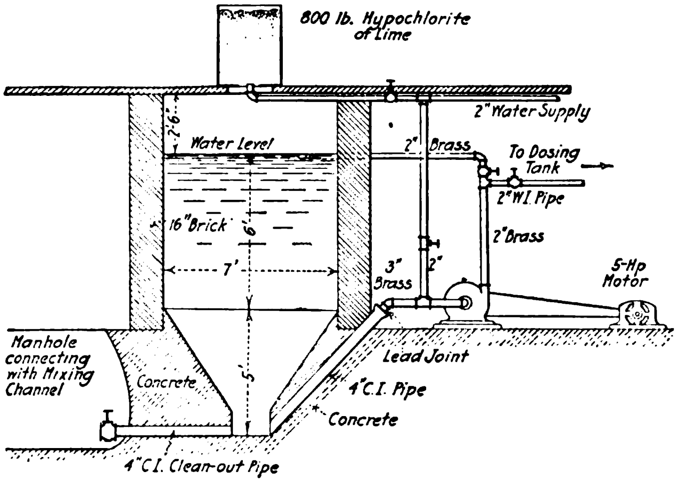


FIG. 209.—Apparatus for preparing disinfectant at Providence.

operatives. The tank, which has a capacity of 2300 gal., is then filled to the usual water level, and the pump started. The liquid is kept in circulation for one hour, after which it is pumped to a dosing tank, where it is diluted to a strength of about 25 lb. bleach per 100 gal. of water, approximately equivalent to a 3 per cent. solution of bleaching powder or a 1 per cent. solution of available chlorine.

One mixing tank only is provided, the bleaching powder solution being pumped from it to two dosing tanks, 8 ft. wide by 20 ft. long by 3½ ft. in depth, holding about 100 gal. per inch in depth, equivalent to 25 lb. of bleaching powder. This makes it easy for the operatives to gage the rate at which the solution is introduced into the sewage, by the drop in the level of the liquid in the tank in a given time.

The rotary motion imparted to the contents of the mixing tank by the discharge from the circulating pump causes the solution to be thoroughly mixed, and further stirring in the dosing tank while discharging the liquid has been found unnecessary. A water connection is provided so that the pump, piping and tank can be flushed out after each dose has been pumped to the dosing tank. This reduces the liability of damage caused by the corrosive action of the bleaching powder. After this apparatus had been in use 6 months all parts of the piping and pump appeared to be in good condition and practically unaffected by the bleaching powder with which they had come into contact. The runner of the pump showed signs of wear from the grinding action of the large pieces of bleach, on account of which it is probable that an iron runner would be preferable to the brass runner.

HYPOCHLORITE PLANTS FOR DIFFERENT CONDITIONS

Brewster, N. Y.—The disinfecting plant at Brewster, N. Y., is on the site of the old electrozone plant mentioned on page 738 and treats the sewage of a large part of the village as well as the water in Tonetta Brook, a somewhat contaminated tributary of the Croton River. There are two solution tanks, 8 ft. high, and 7 and 10 ft. in diameter respectively. These are built of cypress staves and lined with 2 in. of cement mortar plastered on metal fabric. Pipes are provided to drain off the sludge settling to the bottom of the tanks and to feed a porcelain-lined orifice box having two orifices, one furnishing solution to the sewage and the other to the brook.

In February, 1911, the flow of sewage was measured for a period of 30 hours and found to be about 10,000 gal. a day. For disinfecting this quantity, 125 lb. of bleach are used, the bleach being bought on a contract requiring it to contain at least 33 per cent. of available chlorine. This is supplied at a uniform rate although the rate of flow of the sewage is variable. Disinfection is considered necessary because the sewage is carried away by a vitrified pipe line just above the flow line of one of the reservoirs of the Croton water-works system supplying the Boroughs of Manhattan and the Bronx. The brook receives 225 lb. of bleach daily until its flow exceeds 5,000,000 gal. per day, when the charge is increased enough to provide about 1.8 parts of available chlorine.

Plant on Croton Aqueduct.—One of the largest hypochlorite plants is on the Croton Aqueduct of the New York water works at Dunwoodie, where provision has been made to dissolve 3 tons of bleach daily, if necessary. The plant was designed under the direction of I. M. de Varona, and consists of a wooden feed tank on a trestle, and 2 reinforced concrete solution tanks, which are filled and emptied alternately.

The feed tank has an overflow weir which maintains a practically con-

stant elevation of the surface of the water pumped into it. The rate of flow from the tank is regulated by an indicator valve, graduated to read "gallons per minute."

There is a 5 × 3-ft. charging opening in the concrete top of each solution tank, and there is an opening in the common wall between them, fitted with a bronze weir, which limits the depth of solution in either tank to 6 ft., the excess solution being collected in the other tank. Under the charging opening in the top of the tank is a basket of bronze wire with 3/8-in. mesh, with its rim at the surface of the water. The dry bleach is dumped into this basket. Below the basket is a 3-in. galvanized wrought-iron pipe with perforations along its upper surface, through which water escapes from the feed tank under a head of about 4 ft. These small jets cause currents around and through the bleach and help dissolve it.

In the bottom of each tank there is a drain for flushing out the refuse, a 2-in. perforated pipe through which air can be blown to keep the solution thoroughly stirred, and 2 screened openings into 2 small stilling chambers, provided to store a small quantity of the solution under conditions which will maintain a constant surface elevation free from disturbance by the introduction of fresh charges of bleach and the bubbling of air. Each of these stilling chambers has a graduated orifice outlet through which the solution flows at the desired rate into a 6-in. vitrified pipe leading to the old and new aqueducts.

The methods of introducing the solution into the 2 aqueducts are not the same. The older conduit delivers daily about 80,000,000 gal. only, and a 2-in. vertical pipe with 10 1/8-in. holes has proved satisfactory for introducing the solution. This is mixed with the water by the cross-currents produced by 2 frames containing vertical louvres, inserted in stop-plank grooves in the masonry. The new Croton Aqueduct is able to deliver about 300,000,000 gal. and is so large that there is some difference in the velocity of the water flowing in its center and against its walls. The framework of pipes by which the solution is admitted to several points of the cross-section has accordingly been provided with a pair of deflecting wings which divide the current like the bow of a boat, and can be moved so as to present a considerable range of angular obstruction to the flow. This has been found to give a satisfactory distribution of the solution through the water.

A method of handling the drums of chloride of lime has been worked out here which has eliminated some of the unpleasant features of the chlorination process. A hinged jacket is clamped firmly about the drum; this jacket has 2 trunnions by which the drum is picked up by a traveling crane and carried to a pit having a frame upon which the drum is placed, while its head is cut out. It is then revolved on the trunnions and its contents are discharged slowly through a hopper into a box

fitting closely to the bottom of the hopper. The amount of chemical delivered can be observed by graduations marked on the glass sides of the box. Between the hopper and the box there is a rocking grate, like a stove grate, which is shaken to break up the large lumps of bleach before they fall into the box. The hopper, with the grate and box, is slung from the jacket fixed about the drum, and lifted with it by the crane. The box is provided with a canvas hood at the bottom, which is lowered through the charging opening in the top of either solution tank and the measured charge of bleach is dumped into the wire basket, already mentioned, by drawing out the sliding bottom with which the box is fitted. The hood prevents the escape of dust and fumes of chlorine.

Pennypack Creek Plant.—The disinfecting apparatus of the Pennypack Creek sewage treatment works at Philadelphia is shown in Fig. 210. It was designed to treat the sewage from a district with about 10,000 persons, by W. L. Stevenson under the direction of George S. Webster, Chief Engineer of the Bureau of Surveys. (*Engineering Record*, Sept. 6, 1913.) The mixing tank is built of white cedar; it is 3 ft. in diameter, 2½ ft. deep and lined with cement mortar on wire lath. The wooden cover has a door through which the bleach can be added, and a pipe leads from it to an exhaust fan.

The 1½-in. centrifugal pump which delivers the liquid from the mixing to the solution tanks is placed so low that it does not require priming.

The two solution tanks are constructed like the mixing tank and are 4 ft. in diameter by 6 ft. deep. There is a check valve on the supply line to prevent the solution from flowing back to the pump should the valves on the inlets to the tanks be open when the pump stops running. All tanks, pipes and valves are coated with Minwax clear waterproofing. Each tank has an indicator of the depth of the solution, a sludge drain and an overflow pipe. A floating arm is provided to draw off the solution about 6 in. below its surface, where it is believed to be most clear. The solution passes into a constant-head orifice tank, which delivers it as shown in the illustration. It will be observed that this plant furnishes a solution of but one concentration, which is not diluted until it is delivered at the manhole, where fresh water is added in the manner indicated in Fig. 210. The operation of the plant was described by Stevenson as follows:

"A drum of bleach is placed beside the mixing tank and under a dust hood connected to the exhauster. With the exhauster running, the proper quantity of hypochlorite is weighed out under the canvas cone. The powder is then emptied into the mixing tank, small quantities of water are added, and the mixture is stirred by the paddles until it becomes a creamy fluid free from lumps. Water is then turned on and the centrifugal pump started to force the strong bleach mixture to the solution tanks. Since the

inlet to the solution tanks is in the bottom, the sludge deposited from prior use is thoroughly stirrred up by the new solution and any available chlorine left from previous lixiviations completely washed out. During the time the pump is running the contents of the solution tank are thoroughly agitated by the incoming water. As all the water used in making the solution passes through the mixing tank, the pump and the pipes, these portions of the apparatus are freed from bleaching powder by the time the solution tank is filled, and in fact have been thoroughly washed out with clean water, which has prevented their deterioration by corrosion."

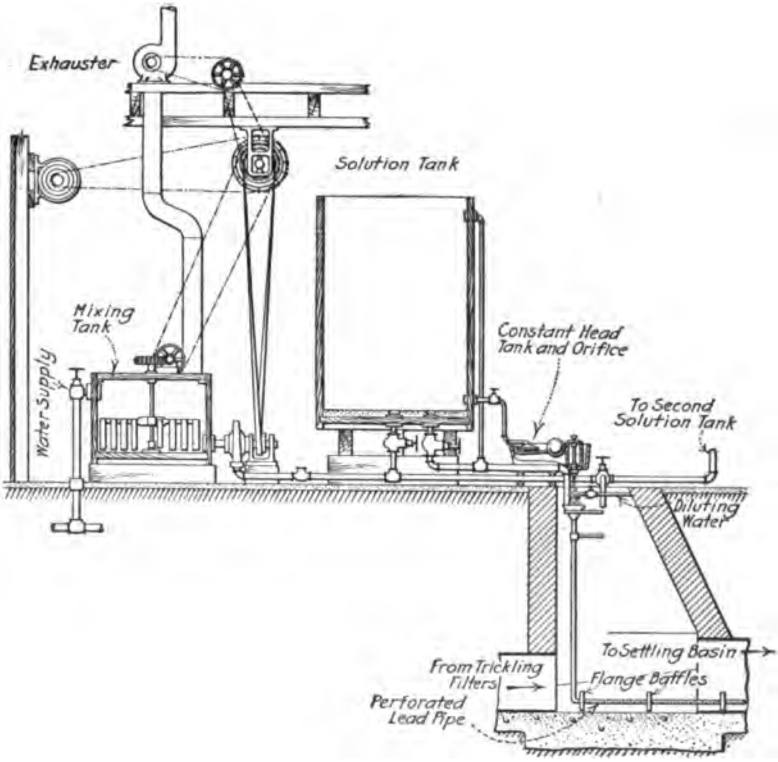


FIG. 210.—Disinfecting apparatus, Pennypack Creek disposal works, Philadelphia.

Emergency Plant.—An example of a very simple apparatus for temporary use is shown in Fig. 211, from *Engineering & Contracting*, July 17, 1912. This is an emergency outfit built by the Indiana State Board of Health from 3 barrels, a commercial constant-level regulating box, a small amount of piping and a geared mixing contrivance much like that used on some types of ice-cream freezers. The New York

Continental Jewell Filtration Co. makes several sizes of portable emergency plants.

Hospital Drain Plant.—The sewage from the Smallpox Hospital at Washington, D. C., is automatically disinfected by an apparatus installed on the house drain by the District Sewerage Bureau, and is discharged into the street sewer by an automatic flushing siphon. The sewage is collected in a concrete chamber large enough to give a 6-hour minimum discharge interval. The disinfectant is placed in a reservoir from which it is drawn automatically into a dosing flask, which is

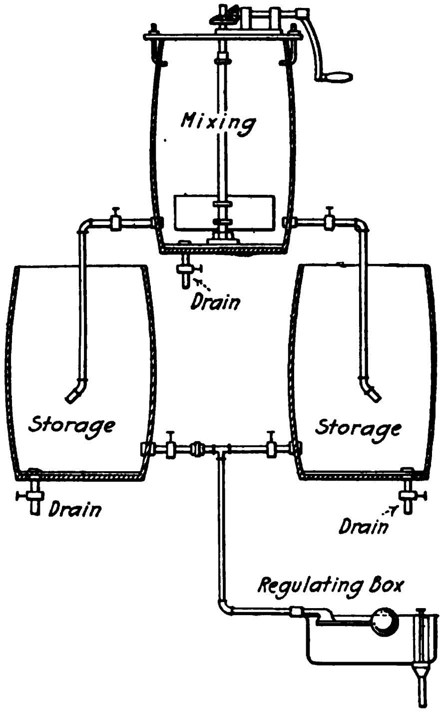


FIG. 211.—Emergency disinfecting apparatus, Indiana State Board of Health.

emptied by the discharge of the siphon chamber. A constant quantity of the disinfectant is applied to a fixed quantity of sewage. There are no moving parts and no metal surfaces about the apparatus, which is constructed entirely of glass to permit the use of any disinfectant.

Institutional Plant.—An apparatus for disinfecting small quantities of sewage and intended to be operated by unskilled attendants, was designed by A. E. Hansen, of the staff of Lederle & Provost, for the treat-

ment plant of the Hebrew Sheltering Guardian Orphanage described in Chapter XXI. For mixing tanks he used single lengths of 30-in. vitrified pipe fitted with concrete bottoms and wooden covers, and for solution tanks, 2 lengths of 24-in. vitrified pipe. These have concrete tops and bottoms and are given several thick coats of paint to make them air-tight. The mixing tanks are connected with the storage tanks by vitrified clay pipe provided with valves of the same material, and have waste pipes for draining off the sludge. It is necessary to keep all valve plugs covered with vaseline in order to prevent air leakage completely. The general arrangement of the appartus is shown in Fig. 230, page 802.

Each storage tank has an air valve in its top and an outlet valve in its bottom. It is filled by closing the outlet valve and opening the air valve and the cock on the supply pipe, and in operation the tank is completely filled until all air is expelled before the air valve and cock are closed. The outlet valve discharges normally below the level of the solution in a constant-level feed box supported below it on wedges. This box is cast iron, enameled inside, and has a 1/8-in. hole at one end near the bottom, which serves as an orifice through which the solution drops into a pipe leading to the place where the sewage is disinfected. The head on the orifice can be changed by moving the wedges which support the box. As soon as enough solution has passed through the orifice to unseal the end of the outlet pipe from the solution tank, a few bubbles of air will pass into the tank and start the liquid flowing from it until the outlet pipe is again sealed and equilibrium is restored, just as is done in many of the water coolers used in office buildings.

Power-driven Mixing Tank.—The mixing tank at the Kansas City, Mo., water works is designed to crush the lumps of bleach while stirring them. It is 3 ft. in diameter and 3 1/2 ft. deep inside, with a vertical power-driven shaft in the center. On the bottom of the shaft is a paddle 2 1/2 ft. long, with cast-iron rollers 4 in. in diameter and 6 in. long attached to it so as to just clear the bottom of the tank. The bleach is ground and dissolved until a creamy paste is produced, which is drawn off at a considerable distance above the bottom so as to reduce the danger of lumps entering the outlet pipe, which has to be cleared out quite often in spite of this precaution.

COSTS

The cost of hypochlorite treatment on a large scale may be estimated from the following notes from a paper by F. D. West before the American Water-works Association in 1914:

"Chloride of lime costs us from $1.22 to $1.70 per 100 lb.; the usual quotation was $1.34, and the average figure $1.40. Taking $1.40 as a basis, we used during 1913 an average of a little over 1200 lb. a day, or $16.80

a day for powder. Two laborers, at 25 cts. per hour, were employed for 8 hours, or $4 per day; making a total cost of $20.80 per day, exclusive of repairs, sample collecting or laboratory analyses.

"One hundred and eighty pounds of liquid chlorine (the amount used April 10) would cost, at 10 cts. per pound $18 per day. We have now passed the worst condition of the year—February and March—when we used 234 lb. a day, or $23.40 cost. It is expected that we will be able to reduce the amount of liquid chlorine to at least ¾ lb. per 1,000,000, or 120 lb. a day. Some supervision and handling of cylinders is required. At present the work is done by a $3-a-day mechanic who also keeps the pre-filters in repair. His wages are charged against the pre-filters. A charge of $1 per day would be fair for this service. This is partly balanced by the discontinuance of laboratory analyses.

"The labor cost[1] during 1913, of $4 per day at Torresdale, with its output of 180,000,000 gal., amounted to but 2.2 cts. per 1,000,000 gal. At Belmont and at Queen Lane the labor cost of about $1.50 per day amounted to 3.8 cts. and 3 cts., respectively. At Roxboro plants the labor cost averaged over $1 per day for mixing; this at Lower Roxboro cost 10 cts. per 1,000,000, and at Upper Roxboro 6.7 cts. per 1,000,000.

"The cost per 1,000,000 gal. at these plants during 1913 amounted to 16 to 18 cts. At 1 lb. per 1,000,000 gal. for liquid chlorine, the cost would be 10 cts., or a saving of 6 to 8 cts. per 1,000,000 gal. On April 14 the quantity used was reduced to ½ lb. per 1,000,000, or a cost of 5 cts., a saving of 11 to 13 cts. per 1,000,000. Belmont and Queen Lane are saving a labor cost of 3.8 and 3 cts. per 1,000,000 gal. Belmont is operating at a rate of ½ lb., and Queen Lane at ¾ lb., or about 5 and 7.5 cts.

"On April 21 the amount used at Torresdale was reduced to ¾ lb., or a cost of $13.50 per day, exclusive of a possible charge of $1 for labor."

TABLE 183.—COST OF DISINFECTING TRICKLING FILTER EFFLUENT AT PENNYPACK CREEK WORKS, PHILADELPHIA (STEVENSON)

Date, 1913	Dry bleach, lb. per 1,000,000 gal.	Available chlorine, parts per 1,000,000, added to effluent	Bleach, cost per 1,000,000 gal.	Bacteria per cc. in treated effluent, agar, 24 hours at 37°C.		
				Total	Acid formers	Like B. coli
Jan. 22–30	92.6	3.8	$1.36	......	......	......
Feb. 1–6	54.5	2.2	0.80	7	0	0
Feb. 7–11	42.9	1.8	0.63	3	0	0
Feb. 12–13	33.5	1.4	0.49	6	0	0
Feb. 14–Mar. 31	23.4	0.9	0.36	38	4	2
April 1–2	21.0	0.9	0.34	12	2	0
April 3–30	20.5	0.9	0.34	14	1	0
May 1–21	19.8	0.84	0.32	10	0	0

[1] The 1913 figures are for sterilization with hypochlorite.

Experience at the Pennypack Creek sewage treatment works, page 770, indicates that there may be a great range in the cost of disinfecting the effluent from trickling filters receiving Imhoff tank effluent. This may be due to changes in the quality of the effluent, to a difference in the character of the attendance on the disinfecting plant, or to a lack of experience to enable the operators to determine readily the minimum safe amount of bleach to use. When the plant was started, the cost of bleach for disinfecting the effluent thoroughly was $1.36 per 1,000,000 gal., according to Stevenson, in *Engineering Record*, Sept. 6, 1913, which cost was gradually reduced to a quarter of that amount, as shown in Table 183.

LIQUID CHLORINE

The use of hypochlorites was found to be attended with a number of disadvantages. Unless the apparatus is designed to prevent the escape of disagreeable gases and care is taken in its use, there may be very unpleasant odors in its vicinity at times. The rapid deterioration in the quality of the chloride of lime after the package containing it has been opened is a decided drawback with small installations, and the irregularity with which the "available chlorine" is added to the water or sewage has proved to be unexpectedly great, even in large, carefully operated plants like the Torresdale filters of the Philadelphia water works. For these reasons, among others, liquid chlorine has come into use in water works, and is now (1915) being introduced in a few sewage treatment plants.

The first use of liquid chlorine for disinfecting water was probably made by Major C. R. Darnell, U. S. A., in 1910. Dr. George Ornstein experimented with it in 1912, and in that year it was also tried at Philadelphia by S. M. Van Loan, at Wilmington by John A. Kienle, at Brooklyn by D. D. Jackson and at Niagara Falls by H. F. Huy of the Western New York Filtration Co. Many plants have been installed since then; each of the 5 filter plants of the Philadelphia water works is equipped with them, and the experience gained there was summed up by West in his paper read before the American Water Works Association in 1914, as follows:

"While in some instances liquid chlorine may prove more costly than chloride of lime, the regularity with which it can be applied, the more effective action on pathogenic bacteria, the small, compact apparatus and the absence of the odor of chlorine around the plant recommend it as a satisfactory substitute for hypochlorite, having, as it does, all the advantages of the latter and only some of the faults."

The apparatus used at Philadelphia was supplied by the Electro Bleaching Gas Co., and consists of a manifold to which 4 to 8 steel

flasks of liquid chlorine are connected by flexible copper tubes, a gage on the manifold to indicate the initial pressure, a regulating valve to reduce this pressure to about 15 lb. per square inch, a second regulating valve to control the pressure under which the gas is distributed, which is shown by a low-pressure gage calibrated to indicate the rate of flow, a hard-rubber pipe leading to hard-rubber absorption towers filled with coke and receiving a small amount of water to absorb the chlorine, and a rubber pipe conducting the chlorinated water to the point of discharge.

The company which supplied the equipment mentioned makes two types, one manually controlled and the other automatically. The

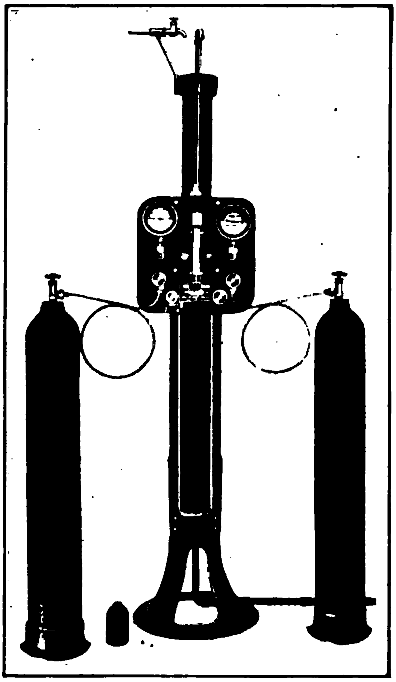


Fig. 212.—Liquid chlorine apparatus (Electro-Bleaching).

former is recommended for disinfecting sewage and is shown in Fig. 212. The steel flasks hold about 100 lb. of chlorine, and are provided with valves for regulating the flow of chlorine, which are constructed to prevent a sudden or large flow of gas being turned on by an ignorant or careless attendant. At the Torresdale water filters it has been found that 1 lb. of liquid chlorine furnished by such apparatus produces the same results as 6 to 7 lb. of chloride of lime, but West con-

siders that with careless handling and storing of bleach at small plants, 1 lb. of liquid chlorine would be found equal to 8 lb. of the powder.

Another type of chlorinator, made by the Leavitt-Jackson Engineering Co., is shown in Fig. 213. The flask of chlorine is suspended from

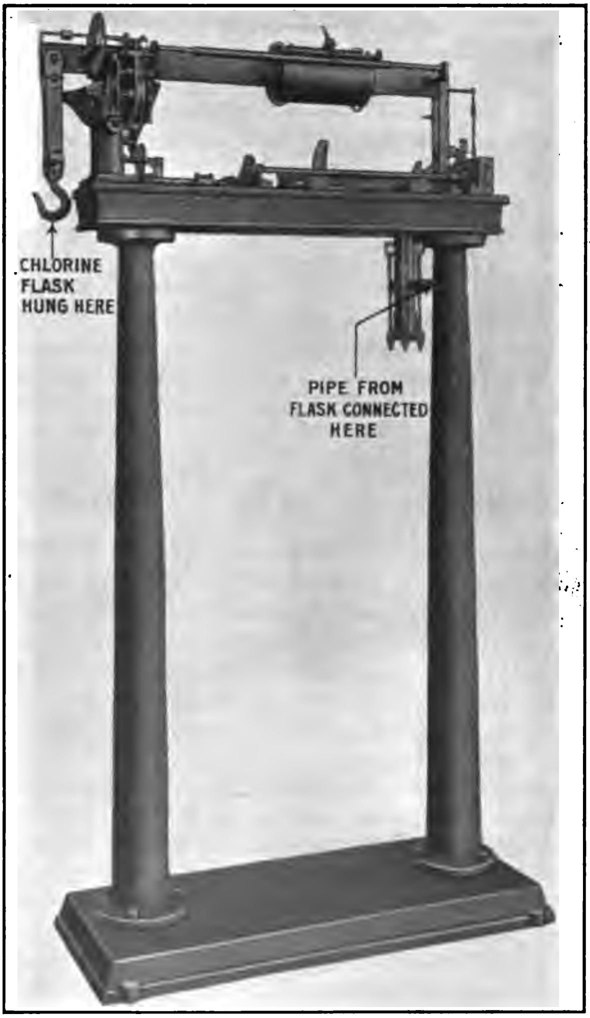


Fig. 213.—Liquid chlorine apparatus (Leavitt-Jackson).

the end of a scale beam and is counterbalanced by a traveling weight which is moved along slowly at a constant rate or in proportion to the

flow of the water, according to the type of apparatus employed. The slightest change in equilibrium of the system of levers, due to the weight being fed forward at the required rate along the beam, causes the regulating valve to open or close until the size of the orifice through which the gas escapes becomes such that the exact amount of gas, by weight, escapes from the machine, keeping the beam in balance. The flask containing the chlorine is not removed when its contents are low, but is recharged from a 2000-lb. cylinder. The latter size is used directly on machines intended for treating sewage.

Where liquid chlorine is used great care should be taken to have good ventilation, for chlorine vapor is liable to injure persons in a room containing it, even if it is present in only small quantities. It interferes with respiration and causes a violent cough with hemorrhage. The nerve centers are affected enough to cause stupor in some cases. Fortunately fatal injury is unlikely. The treatment of a person who has been affected by the gas is merely fresh air and inhaling ether to relieve pain, unless symptoms of acute bronchitis, narcotism and enfeebled heart action are observed, which require also the usual treatments for such conditions. ("Medical Chemistry and Toxicology," Holland, page 121.)

COMMERCIAL HYPOCHLORITE APPARATUS

Hypochlorite disinfection of water developed very rapidly in the United States as soon as information was published regarding the results of Johnson's treatment of Bubbly Creek in the Chicago Stock Yards in 1908, and Fuller, Leal and Johnson's treatment of the Boonton reservoir of the Jersey City water works in 1909. By the end of 1910 the value of the treatment was so well recognized that the standardization of mixing and charging apparatus was undertaken by a number of manufacturers.

For dissolving the bleach, the New York Continental Jewell Filtration Co. makes a cast-iron mixing tank, with a porcelain lining if desired. Directions for dissolving bleach are given by Hooker in his "Chloride of Lime in Sanitation." He considers it important to conduct the process so that the undissolved sludge of hydrated lime and silica will settle rapidly, and for this purpose he advises paying special attention to two points. The first is to avoid making the paste too stiff, for a gelatinizing action will then take place and great difficulty will be encountered in settling; Hooker states that less than ½ gal. of water to 1 lb. of bleach should never be used. The second point is the lack of any necessity for breaking up the lumps of bleach in a thorough way, because the available chlorine nearly all dissolves readily and too much agitation is detrimental to prompt settling.

The New York Continental Filtration Co. recommends discharging the strong solution by gravity into reinforced concrete solution storage tanks in the case of large works, but it makes portable plants with one or two steel storage tanks. Wooden tanks have also been employed. It also recommends adding enough water to the solution tanks to keep the mixture at 2 per cent. strength, as this concentration is considered to have the greatest stability. The solution tanks it constructs, like those of other manufacturers, are provided with mechanically operated paddles to keep their contents well stirred.

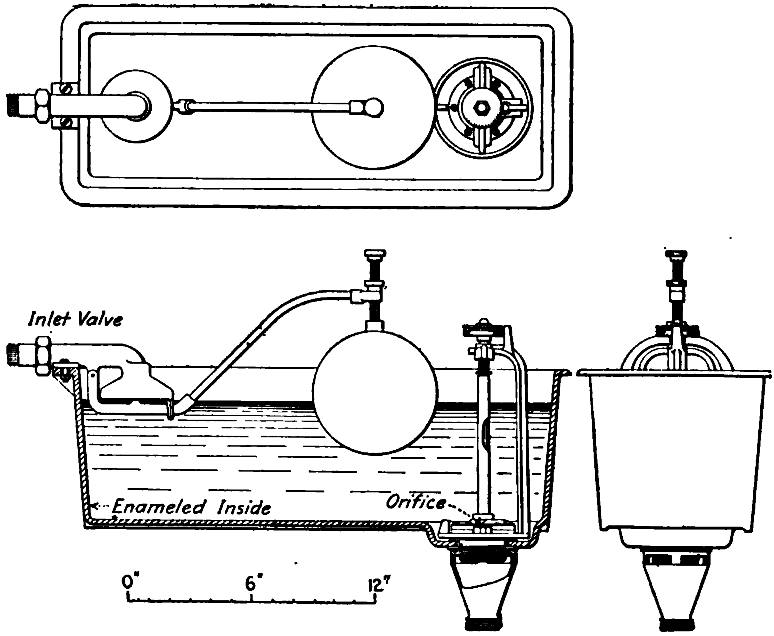


Fig. 214.—Orifice feed tank (Pittsburgh).

From the solution tanks, the liquid is discharged into a chemical feed tank, of which several types are made. That recommended by the Pittsburgh Filter Mfg. Co. for small sewage treatment works is shown in Fig. 214. The ball-valve maintains a practically constant level of the solution in this tank and it is fed at a uniform rate through the orifice-controlling device. There are a number of different types of these devices, three of which, made by the Norwood Engineering Co. are shown in Fig. 215. One consists of a series of round openings of different sizes, one of a taper orifice with a micrometer adjustment at the top and the third is a slotted orifice ⅛ in. wide, cut on a circle and regulated with an overlapping sector. Owing to the very small size

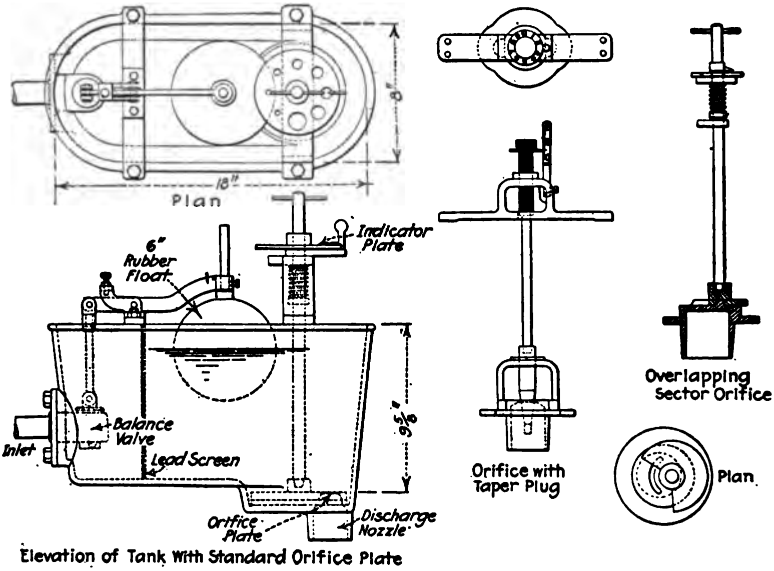


FIG. 215.—Feed tank and orifice-controlling devices (Norwood).

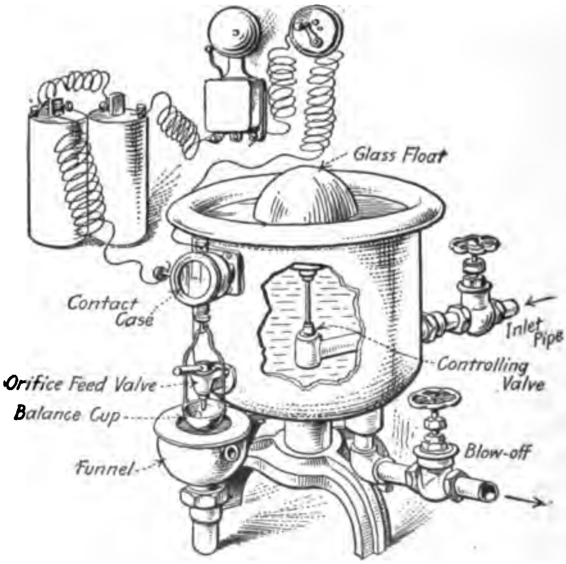


FIG. 216.—Orifice feed tank (Roberts).

of the orifice, it must be inspected frequently in order to be certain that it is not more or less clogged by particles of bleach or scum from the mixing tank. Neglect to attend to this feature of operation is the cause of many failures of sterilizing plants.

At the sewage treatment plant of the Hillside Home, at Clark's Summit, Pa., the orifice feed tank, furnished by the Roberts Filter Mfg. Co. and shown in Fig. 216, is cast iron with a porcelain lining. It is provided with an electric alarm for notifying the operator when the flow of the solution drops too low or ceases. A glass float with a vertical stem operates a controlling valve on the end of the inlet pipe. The solution is discharged through an orifice feed valve capable of close adjustment, and falls into a balance cup held by a light spring, which closes the circuit in a contact case whenever the rate of flow of the solution falls below a predetermined amount. The solution passes from the balance cup into the cast-iron funnel with an enamel lining, and thence by gravity into the sewage.

In two cases where there is a wide variation in the flow of the liquid to be disinfected, a proportionate feed apparatus has been installed. Its general arrangement for feeding chemicals to water filtration plants is shown in Fig. 217. It is used in connection with a Venturi meter, and consists of a master controller and as many chemical controllers as there are solutions to be fed. Where only one solution is employed, the chemical controller and the master controller may be incorporated in a single unit, according to the makers, the Norwood Engineering Co., of Florence, Mass. The master controller has a diaphragm with a connection above and below it to the throat and the upstream end of a Venturi meter. These connections are so made that the resultant difference in pressures between the throat and the upstream end of the meter will tend to raise the diaphragm. The first diaphragm is connected with a diaphragm at a higher elevation, in the base of what is termed the "head tube," by a 1½-in. pipe. The head tube is filled with water, and the weight of this water counteracts the differential pressure applied to the lower diaphragm. The upper diaphragm is connected by a rod with a lever which operates a pilot valve, through which water is supplied or released from the head tube in order to maintain equilibrium. The weight of the moving parts is balanced by a counterweight so that the water level in the head tube will vary exactly with the difference in pressure between the throat and upstream end of the meter. In case the meter has too great a range to permit this ratio of the changes in head to be unity, the diaphragms may be designed to reduce the ratio proportionately to any convenient range in the head tube.

The head tube is connected with each chemical controller by means of a pipe. The chemical controller comprises an orifice tank, an inner

water-level tube, a diaphragm and an orifice. The water level in the inner water tube is the same as that of the water in the master controller, and presses downward on the diaphragm of the chemical controller. The chemical solution in the orifice tank presses upward on the bottom of the same diaphragm. This diaphragm operates a balanced valve on the chemical solution inlet, and in this way the equilibrium of the pressures is maintained. The weight of the moving

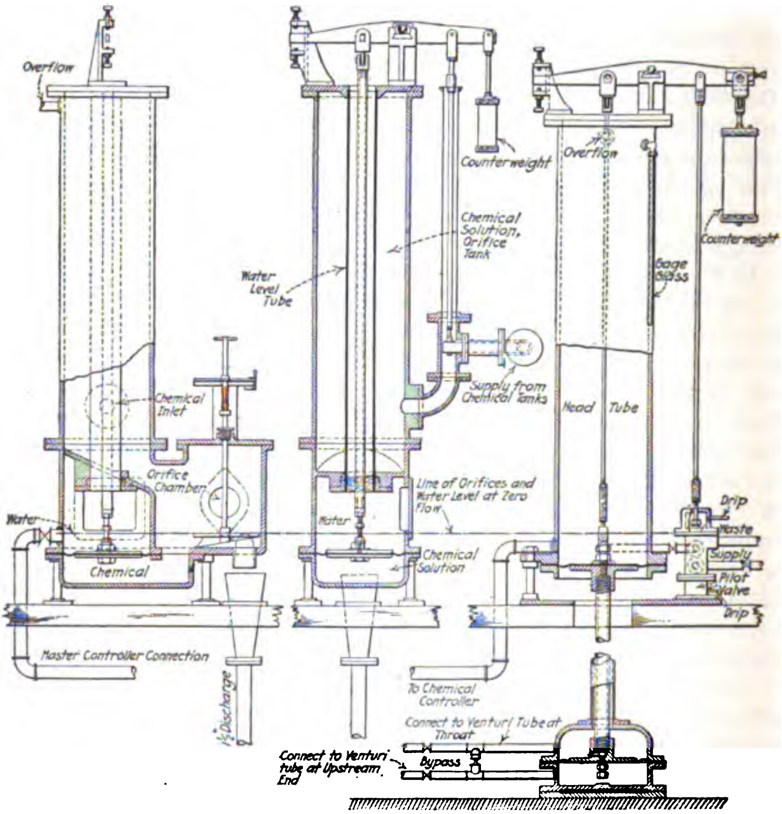


FIG. 217.—Controlling devices for proportionate feed (Norwood).

parts is balanced by the counterweight, and the head of the chemical solution over the orifice will remain in equilibrium with the head in the water-level tube of each controller and that in the head tube of the master controller. As the discharge of the chemicals varies as the square root of the head over the orifice and as the water passing through the Venturi meter varies as the square root of the head difference at the meter, the solution will be discharged in direct proportion to the amount of water pumped through the meter.

CHAPTER XX

DISPOSAL OF RESIDENTIAL AND INSTITUTIONAL SEWAGE

The principles which should govern the disposal of the sewage of residences, estates and institutions are the same as these underlying successful work on a larger scale. In applying the principles, however, allowance must be made for certain factors which influence the design of small plants. In the first place, the amount of sewage is discharged at rates which fluctuate widely, not only from hour to hour of the same day but also from day to day during a week. Where the desired degree of purification of the sewage is high, and the treatment involves methods of filtration which should proceed at fairly regular rates, it is evident that the storage of sewage, so as to permit fairly uniform delivery to the filters and some uniformity in the composition of the applied liquid by mixing the laundry wastes, kitchen wastes and domestic sewage together, becomes particularly important. In the second place, the small size of the plants makes it desirable to have them as nearly fool-proof and automatic as possible. Even if the owner's means render economy in management unnecessary, the importance of automatic operation is great, because experience shows that regular attendance is rarely given to these little plants and they are likely to receive no care until something goes wrong and their existence is indicated in some unpleasant way.

Privies.—The simplest method of providing in a sanitary way for the needs of a small house is by a privy fitted with a pail. Such pails are still used in a few English and German cities, as explained in Volume I, page 13. In building a country privy, care should be taken that the pails are tight, non-absorbent, easily cleaned, inaccessible to flies, and inoffensive to use.

A method of meeting these requirements is shown in Fig. 218. A heavily galvanized garbage pail is used, supported so as to come into contact with the lower side of the hinged seat, forming a fly-tight joint. Flies are kept out of the house as much as possible by a screen on the window and keeping the door closed by a spring. Odors are kept down by using dry, sifted loam or fine ashes as an absorbent. Sand is of much less value for the purpose, and the best material is dry, powdered peat. This pail never becomes soiled on the outside. When it is desired to empty it, the seat is raised, the pail covered with the regular cover sup-

plied with garbage pails, and it is carried away by a man. In the case of the installation illustrated, the contents are buried in a remote part of the garden and the pail rinsed before it is replaced.

Closets of this type are used on the unsewered streets of Moose Jaw, Sask. The pails are made of 24-gage galvanized iron and hold about $2\frac{1}{8}$ cu. ft. A large number were bought by the city in one order at

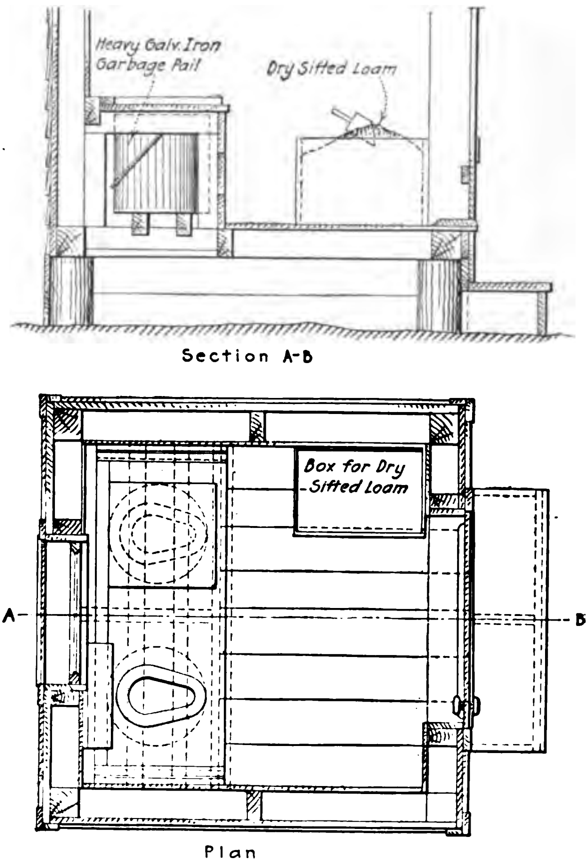


Fig. 218.—A sanitary privy.

$3.05, delivered. They are changed by city employees about every 10 days, and the nightsoil is mixed with garbage and stable manure and burned.

Leaching Cesspools.—The use of leaching cesspools is very common in the United States and has probably resulted in the pollution of a very large number of wells and resulting cases of enteric diseases. These

receptacles are strongly condemned by medical health officers, but their improper use rather than any intrinsic defect is the real source of danger.

A leaching cesspool is merely a dry-laid masonry well, usually without any masonry on the bottom. The sewage flows into it and leaches into the soil. Eventually the solids in the sewage fill the well, and another must be dug. It does not pay to pump out the contents because the surrounding soil is probably clogged. Such cesspools, when sunk in porous soil, are rarely the source of offensive odors. It is desirable to draw in the masonry of the top of the well and provide some sort of cover for the top, like the top of a manhole, and the top 5 ft. of the masonry are often laid in mortar in order to prevent injury by frost.

The liquid contents of such a cesspool are usually in a very septic condition and percolate into the soil at such a depth below the surface that their reduction to stable substances doubtless proceeds in a large degree by putrefactive steps. These conditions explain the nuisance caused by such a cesspool in soil with but little porosity, for concentrated anaerobic changes in sewage are often very offensive. A well-built leaching cesspool in porous soil reduces the danger of odors to a minimum by distributing the liquids of the sewage through the pores of the soil. At the same time, these liquids are a source of danger to any well in the vicinity, and such a cesspool should never be constructed where there is the slightest probability of the passage of the sewage into the region of draught of a well. The cesspool should always be downhill from a well and Theobald Smith gives 100 ft. as the minimum permissible distance between the two. ("Sewage Disposal on the Farm," page 8.) In 1909 the Massachusetts State Board of Health collected and analyzed samples of water from a number of wells in several of the towns which were not provided with public water supplies. At the same time the surroundings of the wells were carefully noted. In nearly all cases the wells were found to be more or less polluted by sewage and in some cases the pollution was so serious that the Board urged the introduction of a public water-supply.

Tight Cesspools.—The notorious cesspools of London and Paris, described in the Introduction to Volume I, were tight and their contents were removed by contractors. Similar cesspools were used in many American cities and their inconvenience and unsanitary nature have been the strongest arguments for sewerage systems. In localities without sewers they are useful, however, when properly constructed and large enough for their purpose. It is manifest that the changes taking place in them are anaerobic and consequently they should be designed on the same basis as septic tanks. The "sanitary sewage tanks" developed by Prof. Robert Fletcher and installed with very satisfactory results under the authority of the New Hampshire Board of Health, are to be classed as well-designed cesspools.

The tank which first suggested the possibilities of such construction, according to a statement by Fletcher (*New Hampshire Sanitary Bulletin*, October, 1908) was an old-fashioned well-shaped leaching cesspool of stone masonry, 8 ft. deep and 6½ ft. in diameter at the bottom. After 21 months' use by a household of 2 to 7 persons, it became filled and its contents oozed out of the joint around the cover. When the tank was opened no solids were found in it, merely a light brown liquid, not overpoweringly offensive to smell. Fletcher built a blind drain about 3 ft. deep and about 40 ft. long, leading down a hillside from the cesspool. He filled the bottom of the trench with cobble stones to a depth of 10 in. and covered them with 3 in. of coarse locomotive cinders, on which a line of 4-in. tile was laid, connected with an elbow inserted in the wall of the cesspool with its inner end turned down so as to be sealed by the liquid in the cesspool when the latter was full. The first three joints of the pipe, numbering from the cesspool, were cemented and the remainder open. Coarse cinders were laid about the pipe, the trench was filled with finer cinders, and soil and sod replaced so as to restore the lawn. The lower end of the pipe ended in a cobble-filled pit, a few feet back from the crest of a steep slope. This method of removing the surplus contents of the cesspool operated satisfactorily.

The first extensive use of the system developed by Fletcher as a result of this experience was made in 1911 at Sugar Hill, N. H., when the lack of sanitary conditions at this important summer resort was brought to the attention of the State Board of Health by the local health officers. In that year 20 cesspools were built from Fletcher's plans, the size of each depending on the number of persons served and an assumed use of 20 gal. of water per person daily. The smallest is 7 × 3.5 × 4.5 ft. and gives considerable storage when used for a house with 4 to 10 persons. They gave such satisfaction that in the next two years 14 more were constructed.

These tanks were constructed of concrete, with tight covers. The inlet enters the tank about 6 in. below the level on which the outlet leaves it and broth are trapped by 2-ft. lengths of pipe running downward. At least 12 in. of clear space are left between the surface of the liquid and the cover, for it was found that with less headroom there was likely to be an accumulation of gas under pressure at times. Observations by the local health office showed that the best results could be obtained only when the house drain was tight, so as to keep out ground water, and the tank was tightly covered. The effluent from these tanks is allowed to trickle over the surface of grass, even in winter, and has caused no objectionable conditions. The warmth of the liquid in the cesspool, which generally has its cover overlaid by at least 1 ft. of earth, seems to be adequate to prevent any continuous freezing of the ground on which the liquid flows.

Where such cesspools are used in towns, it is customary to remove their contents at night, whence their name of "nightsoil." So far as practicable the contents are handled without exposure to the air, by pumping them into tank carts with diaphragm pumps (see Volume II, page 68) of a type made specially for such service. The tanks are emptied into the nearest manhole of a sewerage system, if the house is on an unsewered street of a city having such a system; otherwise they are emptied into bodies of water or into shallow trenches dug in the ground, as the authorities may permit. Such cesspools must be made much larger than those designed by Fletcher, for no overflow of their contents when filled is contemplated.

Prof. Robert Spurr Weston has employed on country estates a pair of cesspools, the first tight and designed to overflow into the second, which is larger and deeper and acts as a leaching cesspool. This combination permits septic action to take place in the first tank and holds back the solids there, so that there is relatively little probability of any clogging of the earth around the second cesspool. The nightsoil in the first tank must be removed from time to time.

An elaboration of this system was used by the authors for the summer school of the Massachusetts Institute of Technology at East Machias, Me. This plant, Fig. 219, was designed for a camp of 200 persons, and has an Imhoff tank from which the sludge is discharged by gravity to sludge-drying beds, while the effluent flows into a leaching cesspool. This arrangement avoids the trouble of cleaning out cesspools.

Subsurface Irrigation.—A means of sewage disposal of particular value for country houses is subsurface irrigation, used extensively in the United States since about 1870. This system consists of a series of pipes laid about 1 ft. below the surface of the ground, with open joints so that the sewage may pass out into the surrounding soil. A settling tank for retaining the suspended matter, and a dosing tank, are essential parts of such a plant. The dosing tank is provided so that the whole pipe system may be filled by the flush of sewage, and thus prevent all the flow trickling out at a few joints as it would if the sewage came in a slow continuous flow. Intermittent discharge also permits the whole of the earth about the laterals to be drained and aerated between doses. The length of lateral pipe necessary is generally estimated at 20 to 100 ft. per person connected, depending on the porosity of the soil. Experience shows that the ground should not be shaded, if the best results are desired. In porous soil, trenches may be dug and the pipe laid directly in them; in other cases the pipe should be surrounded by graded gravel, or the whole area may be underdrained and prepared as a sand filter, the distribution pipe being buried below the surface of the sand, the whole covered with loam and grassed over. These systems are out of sight, generate no odors, and operate satisfactorily in the winter.

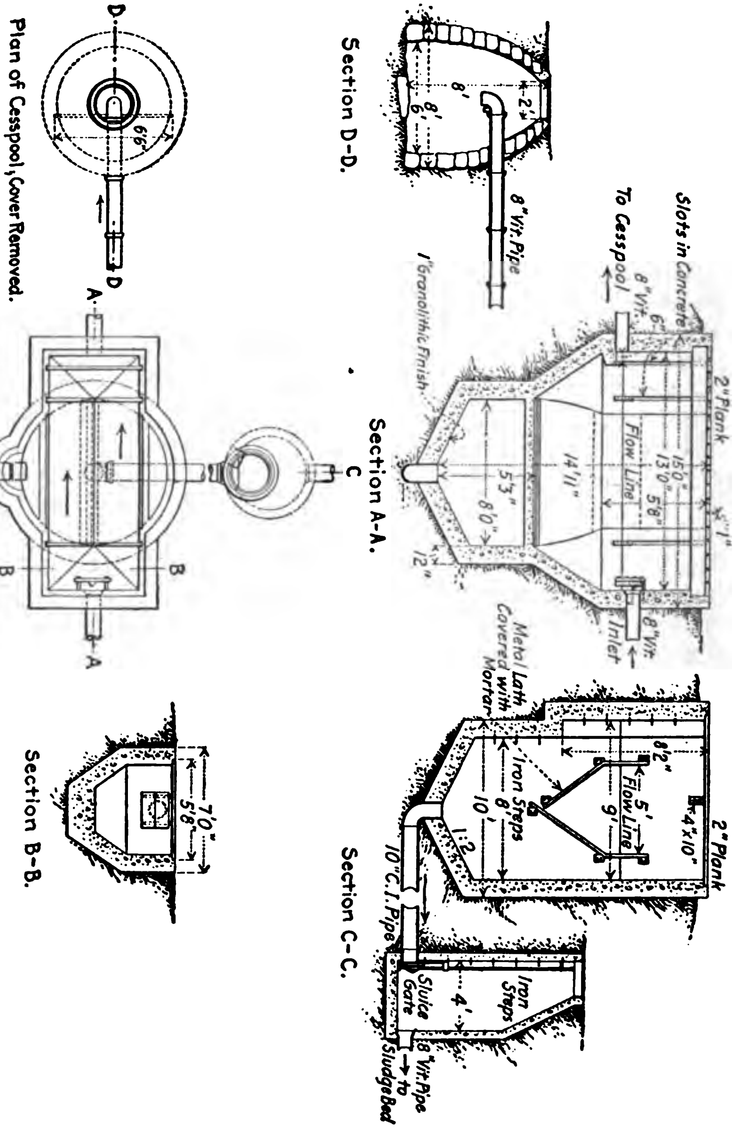


FIG. 219.—Sewage treatment plant at summer school of Massachusetts Institute of Technology.

On the other hand, the grass over the laterals grows rank, the effluent may not be so good as from a sand filter, due to insufficient aeration, and if suspended matter is allowed to pass from the settling tank into the pipe system, the latter may have to be dug up and relaid after a few years.

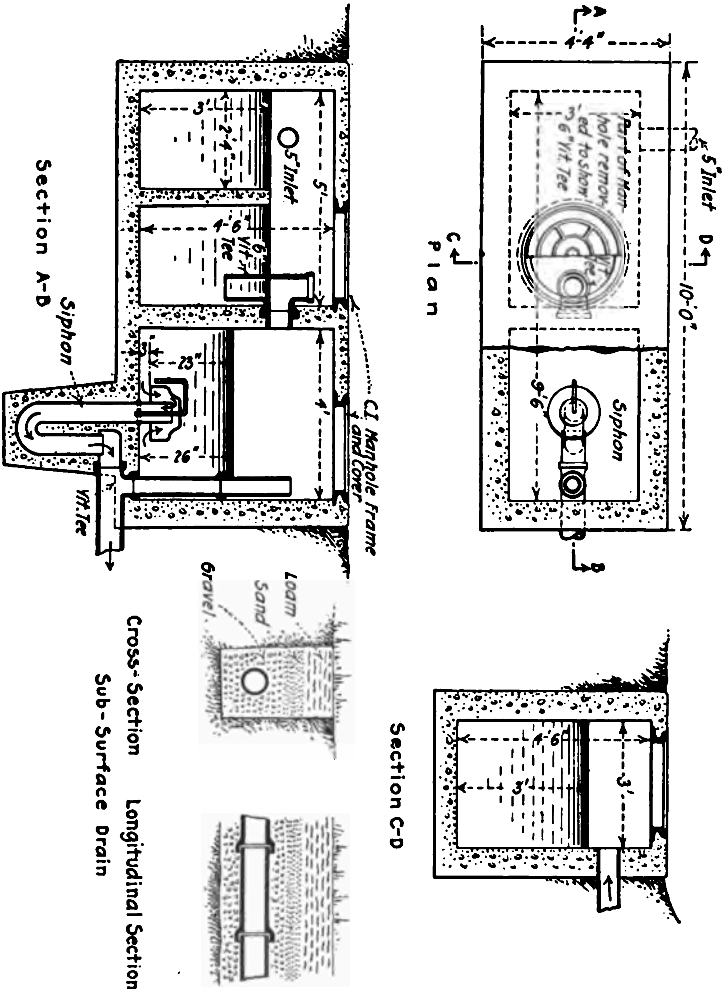


Fig. 220.—Dosing tank and drains for sub-surface irrigation.

While this may not be serious in the case of small installations a system of this kind with efficient settling basins should remain in operation for many years. Particular attention should be paid to preventing floating grease and other scum from leaving the tank. The type of

tank shown in Fig. 220 has been very successful. In this, the sewage coming from the house in sudden flushes stirs up the sediment in the first chamber more or less, but trickles over the dividing wall and enters the second chamber very quietly.

The proper slope of the lateral pipes depends somewhat on the porosity of the soil. In very porous material it should be greater than in more impervious soil, so that some sewage may run clear to the lower

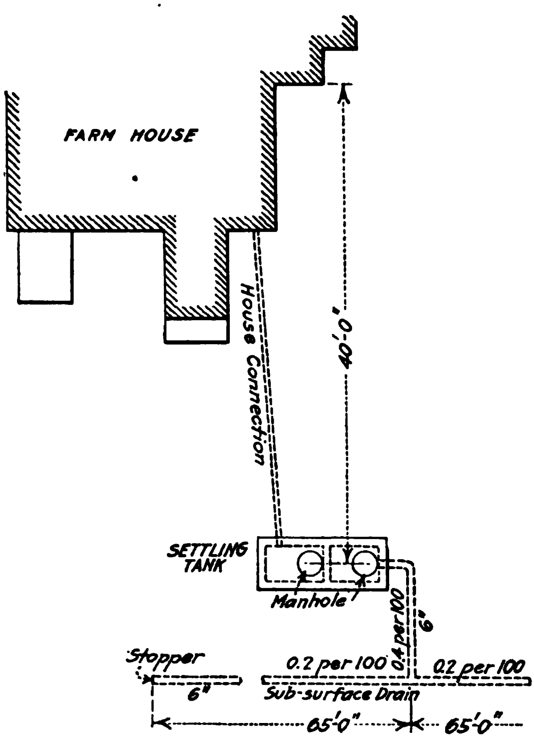


FIG. 221.—Subsurface irrigation for farm house.

end of the system and not all flow out of the first few openings. In general the slope should be from 2 to 5 in. per 100 ft.

In Fig. 221 is shown the authors' design of a plant for a farm house with 6 to 12 occupants. This work complete cost about $210, and has given rise to no complaints of odors or stoppages.

There is occasionally trouble in making the siphons of small dosing tanks stop discharging promptly, instead of which they dribble the last part of the dose. This happened at the dosing chamber of the trickling filters at the Montefiore Home at Bedford Station, N. Y. The plant

was designed by Geo. W. Fuller and the remedy for the poor siphonic action was described by him substantially as follows:

Float boards were employed for this purpose. Each board was a 1 × 4½-in. plank across the end of the septic tank as shown in Fig. 222. It had a double strip of rubber on the bottom and ends to keep down leakage. It was hung by padlock hinges from two 1 × 3-in. battens, to the other end of which a 6 × 6 × 18-in. wood float was attached by two 1 × 2-in. hangers 18 in. long. When the sewage rose 3 in. above the crest of the partition wall which originally was the flow line, it began to pour over the top of the float board. It was not long after this before the float began to rise, dropping the scum board and allowing the sewage to pour into the dosing tank and start the siphon smartly. As the sewage was discharged by the siphon, the float dropped and lifted the float board so as to shut off abruptly the entrance of any more sewage into the dosing tank, which resulted in a satisfactory break in

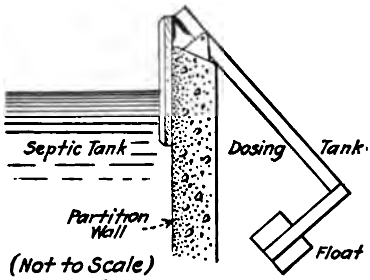


FIG. 222.—Automatic float board for dosing tank

the action of the siphon. Experience has indicated that the float should be made of some material not acted upon so severely as wood, and that glass, concrete or some similar material should be used instead of wood for baffles in small tanks.

Irrigation.—An example of an institutional plant where disposal by irrigation is practised is afforded by the works built in 1914 at the Orange County Hospital and Farm in California, from the plans of Prof. Charles Gilman Hyde. The interesting feature of these works is the Imhoff tank, about 700 ft. from the center of the group of buildings served. This tank is shown in Fig. 223, and was specially designed to meet the conditions of the dry, mild California climate, where the successful operation of septic and Imhoff tanks is rendered difficult by the extraordinary biological action which sometimes occurs, lifting all the sludge to the surface as scum.

In order to ensure thorough distribution of the sewage through the cross-section of the sedimentation chambers there is a baffle with slots

of unusual shape in front of each end wall. Hyde favors the multiple-chamber type of tank because the ratio of depth and width to length in the sedimentation chambers can frequently be made more satisfactory than is possible with the single-chamber type. The control of the inlets is had at an independent manhole. The sewage leaves the tanks over weirs with lips projecting horizontally so that the liquid falls clear of the wall of the collecting chamber, instead of running down the masonry. From the chamber it passes through a suction pipe to either of two pumps, which raises it high enough to give a gravity flow through about 750 ft. of 6-in. vitrified clay pipe to the sandy filtration area. The pump is operated electrically and started and stopped automatically by a controller actuated by a float in a chamber connected at the bottom with the collecting chamber. The tank and pump chamber are underground and the entrance is covered by a pergola, with latticed sides protecting a lining of copper mosquito netting. The sludge is pumped to a drying bed immediately behind the tank.

Intermittent Filtration.—Intermittent filtration is often used for treating the sewage of a very small community, where sand can be obtained at low cost for the beds. This has been done since 1891 at St. Mark's School, Southboro, Mass., with a population of about 225 in 1910. There are 13 small beds and the flow of sewage over them is directed by the head master, who changes the position of the regulating gates twice a day. These beds were not underdrained and the sewage reaches them without passing through settling chambers of any kind.

A somewhat more elaborate system was built from the authors' suggestions in Woodville, Mass. The sewage is stored in a dosing tank, 10 × 12 × 4 ft. deep, from which it is released about twice a week. The dose is delivered through a 6-in. sewer to one of two artificial sand filters with 4-in. underdrains, overlaid by 9 in. of gravel and coarse sand and 3 ft. 3 in. of good filtering sand.

It often happens that sand suitable for intermittent filtration cannot be readily procured, and in this case coke, locomotive cinders and screenings can be used. A plant of this kind built at Pomfret, Conn., from the plans of the authors is shown in Fig. 224. This cost, with 424 ft. of sewer, about $1180. The volume of sewage was estimated at 8000 to 10,000 gal. per day. The sewage first passes into a septic tank holding 8250 gal., giving a detention period of 20 to 24 hours. It passes over the crest of the end wall of the tank to a dosing chamber holding aboug 2200 gal., so that about four doses a day are discharged. These go to two 20 × 50-ft. coke beds, which are flooded to a depth of 3½ in. at each application.

Where a filter must be constructed near the house, R. Winthrop Pratt has suggested that it may be placed underground if there is any probability that it will prove a source of annoyance on the surface.

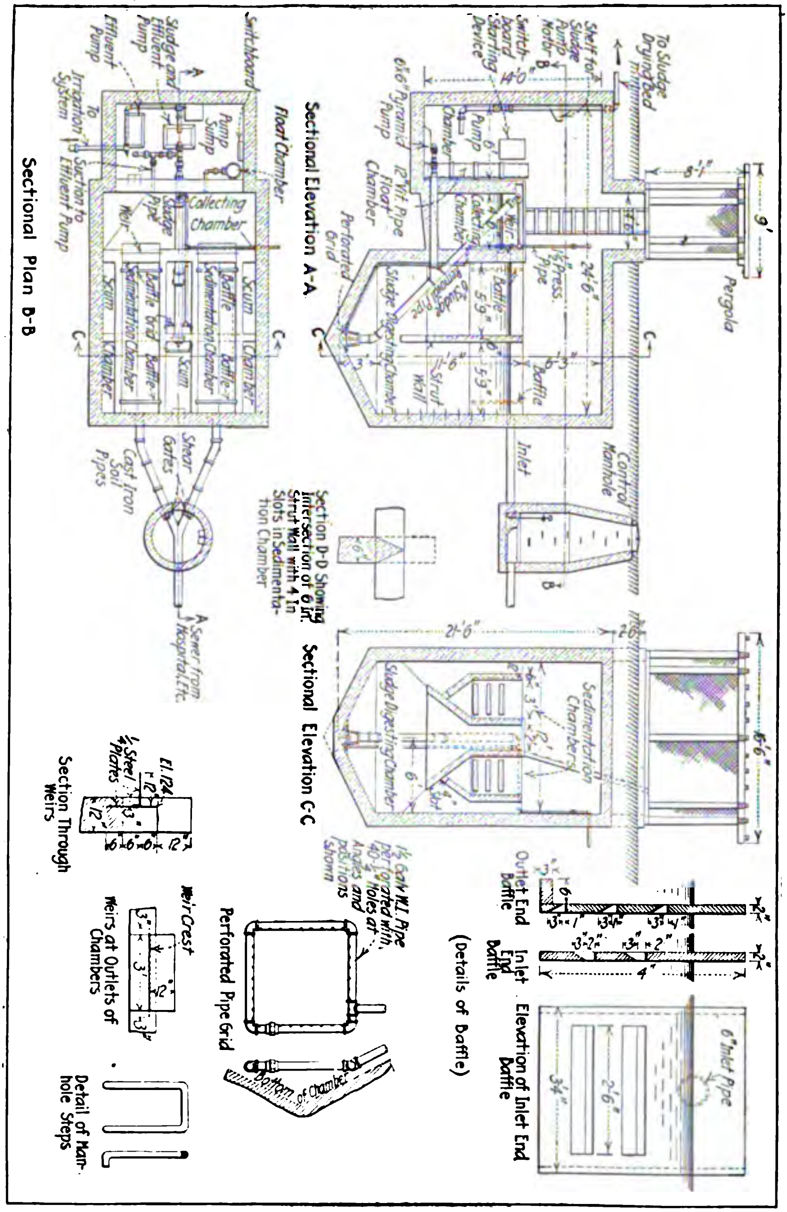


FIG. 223.—Imhoff tank and appurtenances at Orange County, Cal., hospital and farm.

If placed underground, the space above the sand must be well aerated, and even then there is some question as to the extent of the purification attainable under such conditions.

Wherever tanks of any sort are used in residential and institutional plants, the outlets should be trapped or protected by scum boards in such a way that no grease or other floating matter will leave the

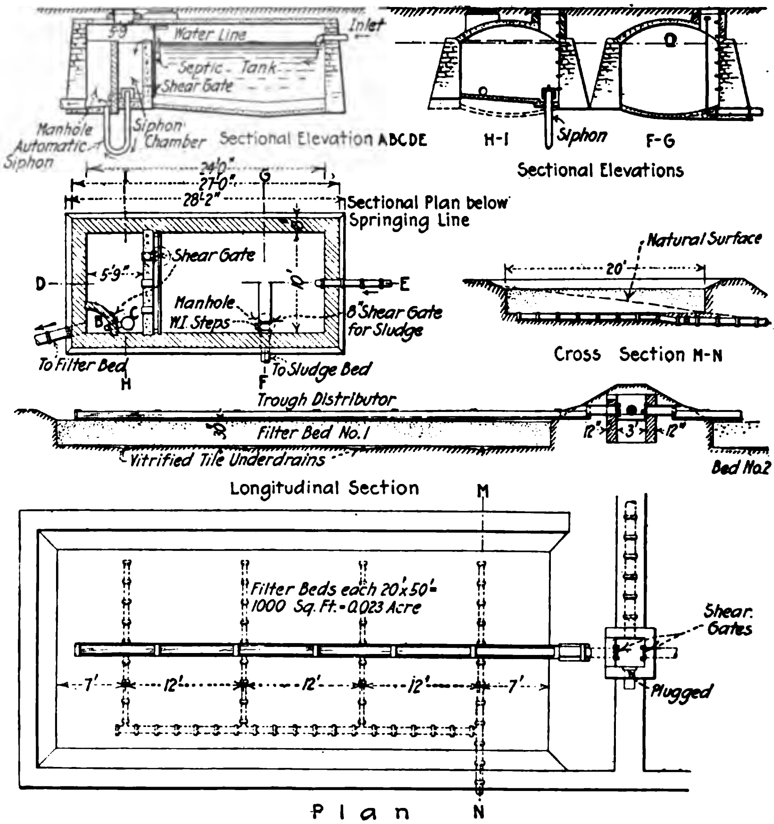


FIG. 224.—Septic tank and coke filter, Pomfret, Conn.

tanks. Such plants rarely receive any attention and if grease and floating solids are deposited on contact beds or filters a nuisance may be created long before any trouble would arise were such materials kept in the tanks.

Contact Beds.—Contact beds have been extensively used at plants for the treatment of the sewage from institutions because they require very little head, can be constructed in many places where filter sand

can be obtained only at prohibitive prices, do not yield the odor which arises when sewage is sprayed over a stone bed, and can be filled from below where the appearance of the works must be given special attention and it is desired to keep the sewage out of sight at all times. In many cases they are used as an intermediate step in a triple scheme of

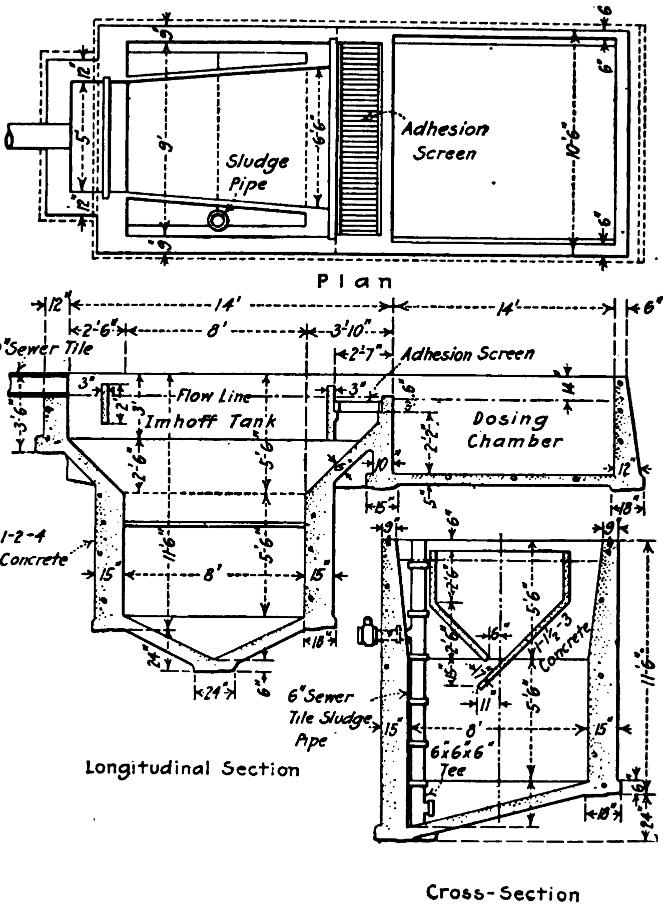


FIG. 225.—Imhoff tank and dosing chamber, Julietta, Ind., hospital.

treatment, as at the Julietta Insane Hospital, Marion County, Ind., described in *Engineering News*, May 1, 1913. The institution had about 200 inmates at the time of the construction of the plant, which is large enough for a considerably greater population.

In this plant the sewage first enters an Imhoff tank of the dimensions shown in Fig. 225, the nominal rate of flow being about 4 ft. per hour.

It has a peculiar detail at the outlet, described by Charles Brossmann, the designer, as follows:

"The sewage flows upward and out through an adhesion screen made of ¾ × 10-in. plank, ¾-in. apart. The top of the screen is submerged about 2 in. below the sewage level. The purpose of this screen is to prevent anything being carried over into the dosing chamber, and thence into the contact beds, and also to serve as an adhesion rack to catch any minute floating material until it has attained sufficient weight to slide off, when it will

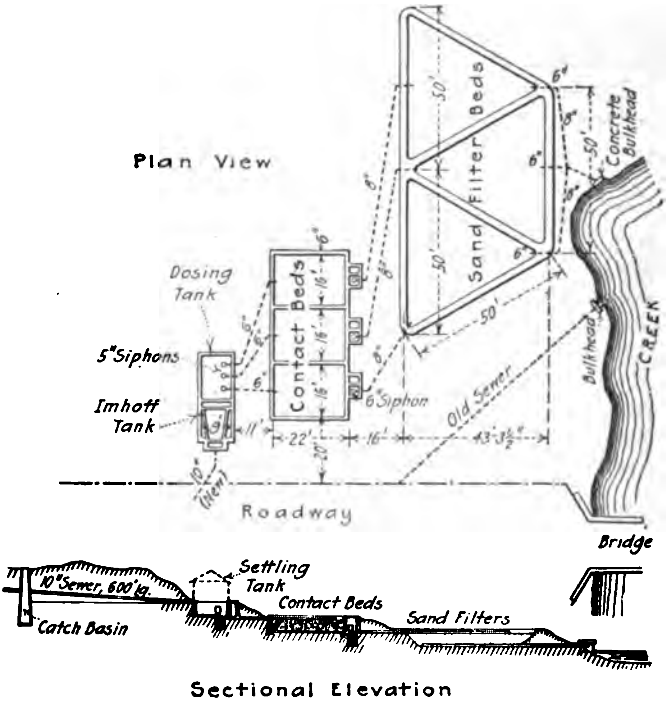


FIG. 226.—Disposal works of Julietta, Ind., hospital.

drop to the sludge chamber below. To what extent this screen will serve its purpose cannot be said until the plant has been longer in operation, but already a considerable amount of matter adheres to it."

This rack is substantially the same in purpose as the colloiders in the Travis tank. The effluent from the tank was stated to contain but little suspended matter and to have only a slight odor, but as the tank is located very near a road, the intention was to roof it over soon. The designer informed the authors in 1915 that the plant had operated satisfactorily in all details, and the adhesion gratings in the outlet of

the Imhoff tank work so well that similar gratings had been used at several tanks built later.

The general location of the parts of this plant are shown in Fig. 226. The dosing chamber has a capacity of 2500 gal. and is discharged by three 5-in. siphons, one for each of the 16 × 22-ft. contact beds. These beds have 33 in. of ½ to 2-in. broken stone, held in place by reinforced concrete walls. The sewage is distributed over the surface by plank troughs, painted with asphaltum. The beds stand full for 2 hours and stand empty for 6 hours. The underdrains are 4 and 6-in. vitrified tile, draining into dosing tanks which are discharged by 6-in. timed siphons.

The contact bed effluent is treated on three filters having 30 in. of sand resting on gravel from ⅛ to 1 in. in size. Trough distributors are used, with notches cut in their sides from 6 to 8 ft. apart to allow the liquid to flow out. Around the outside of these notches the sand is covered with broken stone to prevent any washing away of sand. The beds have vitrified tile underdrains covered with broken stone.

Trickling Filters.—Some very small plants have been built since 1910 in which trickling filters are employed. One obstacle to their use in residential and institutional plants is the head required for their operation. Others are the offensive odors and the little flies, both of which may prove serious drawbacks to this type of filter for residential and institutional installation. Where none of these obstacles is prohibitive, trickling filters afford a good means of treating tank effluent where sand for intermittent filtration is unobtainable or the space available for a plant is restricted, but they do not furnish an effluent so stable and clear as that from a good intermittent filter. This distinction must be kept in mind in designing small plants where the stream receiving the effluent is small and sluggish or the volume of water in the pond receiving the sewage is relatively small.

A plant of this type was constructed from plans by Samuel A. Greeley for serving a country place where from 22 to 28 persons lived. The effluent could be discharged into a ravine, so that a fall of 50 ft. was available, but the house drain was too low to enable surface treatment at the top of the ravine. Space along the ravine could not be obtained for sand filters, and it would be advisable to cover them. The plant selected comprises an Imhoff tank 5 × 4.5 × 13.5 ft. deep and the trickling filter shown in Fig. 227. The sedimentation chamber has a longitudinal baffle so placed that the sewage flows along one side of the chamber from an inlet in the end wall, turns around the end of the baffle at the other end of the tank and then flows back along the other side of the chamber to an outlet in the same end as the inlet. In this way the sewage traverses the length of the chamber twice. On the basis of 50 gal. of sewage per capita daily and 25 persons, the

detention period is 3 hours at the average daily rate of flow. The sludge chamber is designed to hold the sludge of 20 months, and the gas vents have an area of 125 per cent. of that of the sedimentation chamber. Wire-glass was used for the partitions in the tank.

The effluent is delivered to the trickling filter shown in Fig. 227, from *Engineering & Contracting*, December 16, 1914, in which this plant is described in detail. The filter is designed for operation at a rate of 10,000 persons per acre daily. The sewage is applied by the

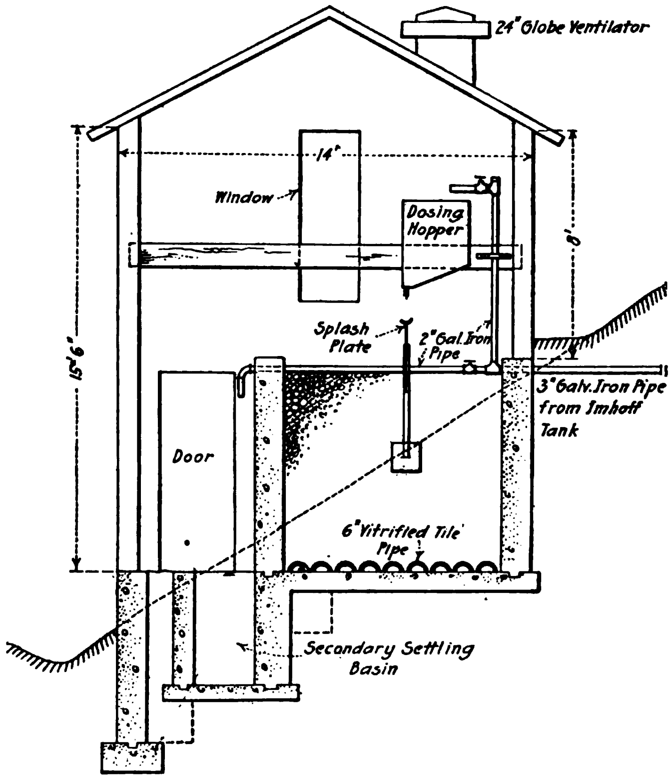


FIG. 227.—Trickling filter for country place.

tipping tray shown in Fig. 228, which discharges 1.5 cu. ft. alternately on one of two brass splash plates, 3 in. in diameter and dished on top to a radius of 4 in. These splash plates were selected to avoid troubles due to nozzle cleaning. Each bucket discharges into a galvanized sheet-iron hopper, shaped to give a varying head and varying quantity of sewage, corresponding to the decreasing annular area of the bed reached by the spray.

The filter contains 6.5 ft. of 1¼ to 2¼-in. washed broken limestone. The underdrains have riser pipes at each corner for ventilation and flushing, and the effluent passes into a basin having a detention period of 1.5 hours at the average daily rate of flow. The entire plant, including 346 ft. of pipe sewer and the house over the filter, cost about $2120.

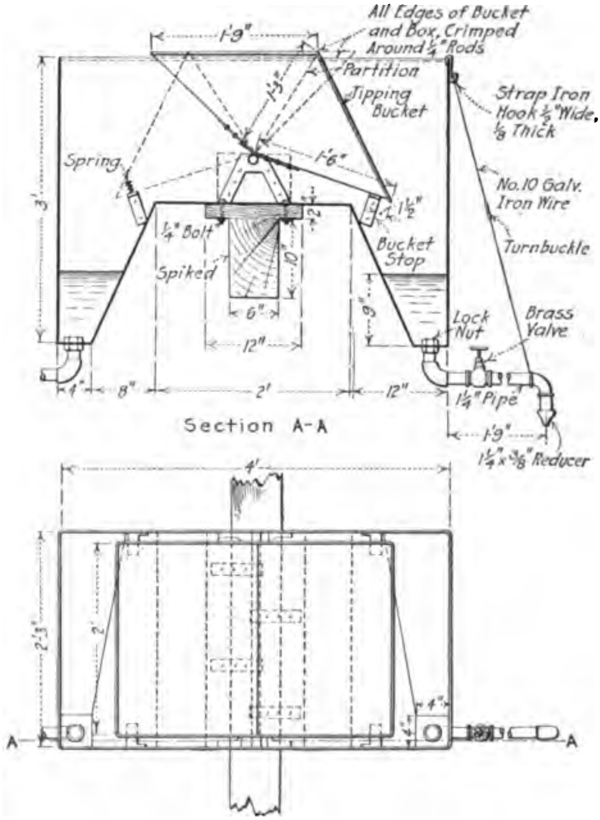


FIG. 228.—Tipping tray for small trickling filter.

Disinfection.—Disinfection is provided for in the sewage treatment plants of a number of hospitals but it is questionable how well they are operated in some cases. As explained in Chapter XIX, disinfection plants using bleach must be carefully and intelligently cared for, or they will probably fail to accomplish their purpose. That purpose may range from the disinfection of merely screened and settled sewage to prevent contamination of the water of bathing beaches or oyster lay-

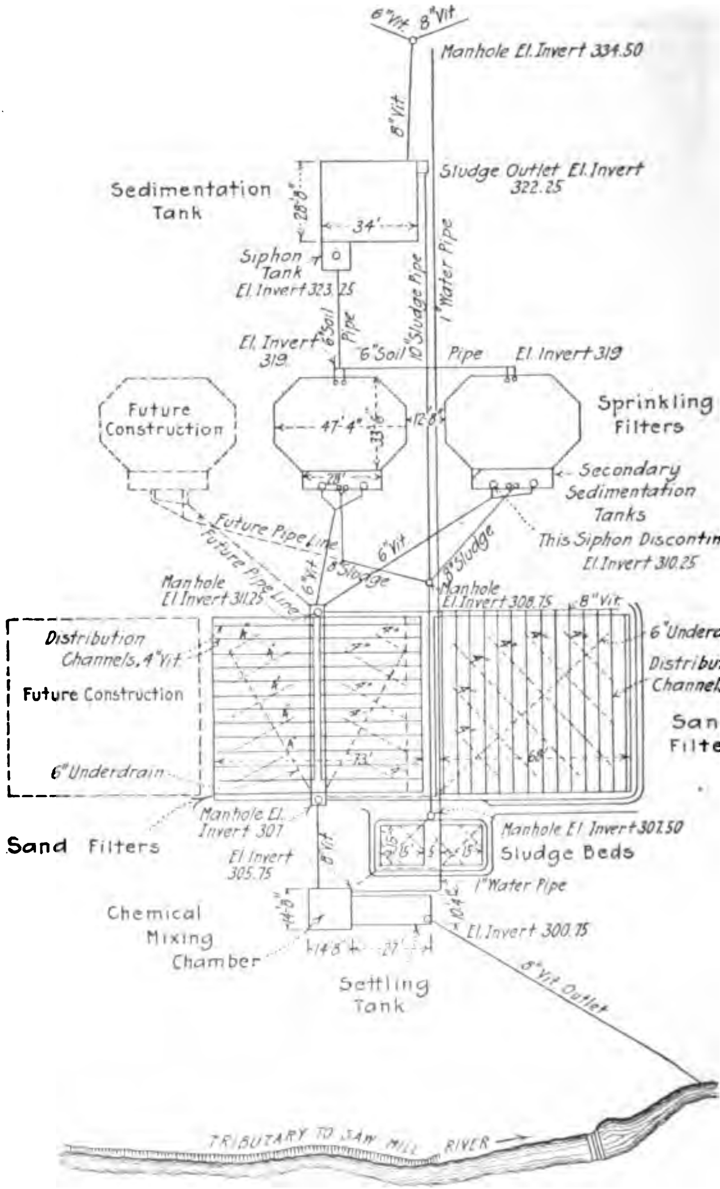


FIG. 229.—Sewage treatment plant, Hebrew Sheltering Guardian Orphanage.

ings, as at Providence, to the production of an effluent meeting the standards for drinking water, as at Pleasantville, N. Y. The plant at the latter place affords not only an example of sewage purification, in a literally true sense, but also a record of unsuccessful operation by incompetent attendants followed by very successful operation under the direction of the designers. The plant will, therefore, be taken up in some detail in this place.

The plant, Fig. 229, was designed by Lederle & Provost to care for the sewage of the orphanage maintained by the Hebrew Sheltering Guardian Society. There are about 30 buildings, with a maximum population of about 1000. The sewage is of a domestic character and its purification is considered of great importance because the entire property is drained by streams which furnish potable water supplies after filtration. The soil is unsuited for any land treatment. The only point along the stream where the sewage from the entire orphanage can be discharged adjoins a residential section of Pleasantville and is only 2000 ft. upstream from an ice pond.

The sewage first flows into the sedimentation tanks, two of 7500 gal. capacity and one holding 15,000 gal., arranged so they can be used in parallel or in series. They are housed over, and all doors, windows and ventilators are screened from May 1 to November 15. Over the center line of each small tank is a 1-in. water pipe with sprinkling nozzles for breaking up and precipitating scum, and there are two such longitudinal pipes over the large tank. The detention period in the tanks is about 7 hours. An independent pipe connects each tank with a 2000-gal. dosing tank having an 8-in. siphon with 5-ft. draft. The flush period is about 8½ minutes. The number of flushes is recorded by a float-actuated Crosby revolution counter, which permits an approximate estimate of the amount of effluent.

The settled sewage is discharged over two 1175-sq. ft. sprinkling filters. Each is divided into halves, independently supplied, and each has 10 Taylor nozzles. Each filter is constructed within a house having concrete walls and a frame roof, the walls having numerous openings slanting downward from the outside to ventilate the bed. The average depth of the stone above the underdrains is 6½ ft. The stone is ½ to 2 in. in size. The underdrains are 4 × 6-in. brick conduits on 12-in. centers, connected to 5 × 10-in. marginal conduits, which discharge the effluent into secondary settling basins, one for each unit. Each is designed to hold the effluent about 2.5 hours when the filters are operating at 1,500,000 gal. per acre daily. Both the inlet and outlet openings are baffled. The outlet delivers the effluent into a dosing chamber, which discharges about 1300 gal. per flush to a group of sand filters.

The sludge is discharged from the sedimentation and secondary

sedimentation tanks upon two 225-sq. ft. superficial areas. Each bed has about 12 in. of steam cinders over the underdrains, which discharge the liquid separated from the solids into the disinfecting house. When the sludge is dry enough for convenient handling by spades, it is removed and buried in trenches.

The sand filters have 36 in. of sand having an effective size of about 0.25 mm. and a uniformity coefficient of about 3. This sand rests on 2 in. of ¼-in. stone, resting in turn on 2 in. of ½ to ¾-in. stone carried

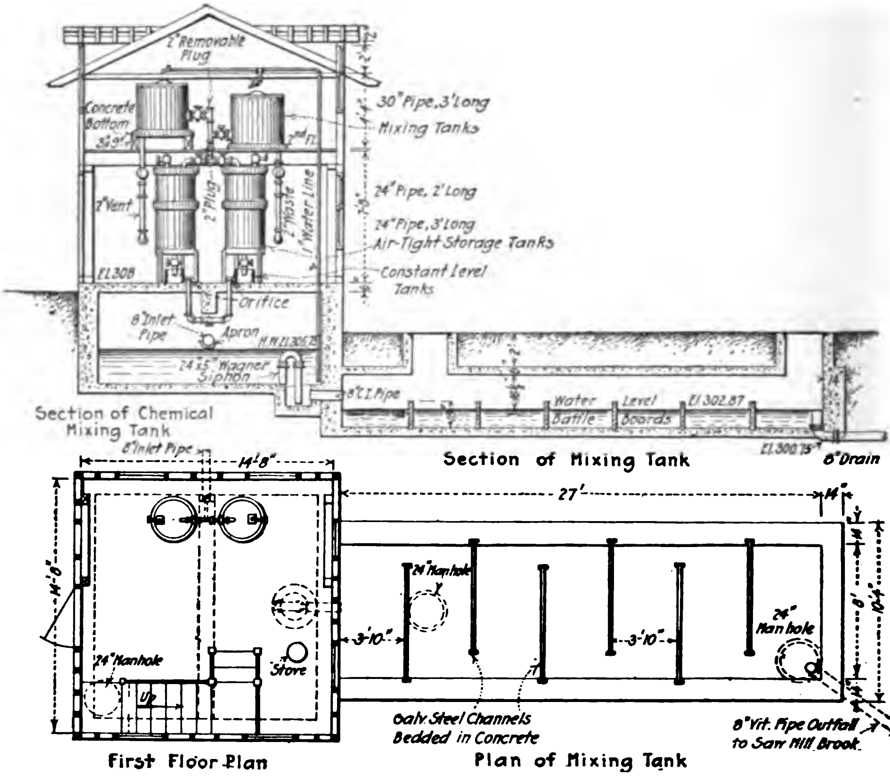


FIG. 230.—Disinfecting plant, Hebrew Sheltering Guardian Orphanage.

by a layer of 2-in. stone in which the 4-in. laterals of the underdrainage system are laid on about 10-ft. centers. The sewage is distributed by lines of channel pipe on 5-ft. centers. There are three of these beds, with a total area of 0.2 acre, and they were designed for operation at a rate of 400,000 gal. per acre daily.

The filter effluent is conducted to the disinfecting plant shown in Fig. 230. This comprises a 2500-gal. well, where the disinfecting solu-

tion is applied, chemical apparatus which was described on page 772, and a covered mixing tank. The house containing the chemical apparatus covers the clear well. It has the duplicate solution tanks and duplicate constant-level feed tanks on the first floor, and the mixing tanks on the upper floor. The principle employed in the operation of the solution tanks does not require any throttling of the discharge pipes, and the rate of flow of the solution is controlled by the size of the orifice in the constant-level tanks and the head on the orifice, the latter being capable of adjustment by wedges under the tanks. The solution drops into the well below the house, from which the dosed filter-bed effluent flows through a mixing tank, shown in Fig. 230, provided with baffles. The detention period of the liquid in the well and tank is about 1 hour.

Careless and Careful Operation Contrasted.—The history of the first 3 years' service of the sewage treatment works of the Pleasantville orphanage, just described, affords a good lesson of the need of intelligent care in operating plants for this purpose. These works were operated successfully for the first 4 months after their completion, under the direction of Lederle & Provost. For the next 6 months the orphanage staff operated them with unskilled labor. The sedimentation tanks were allowed to become highly septic. The ebullition of gas in the tanks carried so much suspended matter into the effluent that the sprinkling nozzles became clogged, causing the sewage to back up in the pipes and the scum in the sedimentation tanks to overflow the outlet wall into the dosing tanks. When the nozzles were cleaned the scum passed to the sprinkling filters and a large amount of solid matter passed from the sprinkling filters to the intermittent filters, which became so clogged that the sewage pooled and froze on the surface. The operation of the plant was manifestly a failure.

At the end of this unsuccessful experience, the designing engineers were again retained to supervise the operation. An attendant was assigned to the work under their direction, and substantially all his time is devoted to it. He keeps a daily record of the condition of each sedimentation tank, each half of each sprinkling filter, each sand filter, each sludge bed, and the bleach used in the morning and the afternoon. The nature of the maintenance work done on each part of the plant is recorded at the same time, and the volume of sewage treated. The expert supervision comprises visits to the plant every few days, methylene-blue tests at each visit and laboratory analyses of the effluent and brook water less frequently. The tanks remain in service an average of 41 days between cleanings.

As thus operated, the settling solids are reduced by 7 hours' detention in the sedimentation tanks from 5 cc. to a trace, when measured in an Imhoff conical glass.

The trickling filters are all operated simultaneously with only the period between doses for resting. This averages about 82 per cent. of the total period of time. The rate of operation is about 1,100,000 gal. per acre daily, two-thirds of the designed rate. The secondary sedimentation tanks retain the filtrate under these conditions about 3 hours; the effluent stands putrescibility tests for 4 to 28 days. Sludge is drawn off about once a month.

Sludge is received by the sludge beds from the sedimentation and secondary sedimentation tanks about twice a month, on an average. Except when frost occurs, the sludge can be handled with spades after drying 6 to 12 days. During the winter, however, it is troublesome to remove sludge for burial, and Lederle & Provost informed the authors in March, 1915, that it would be necessary to provide separate sludge storage and digestion tanks or to increase the area of sludge-drying beds. Although the existing beds are within 250 ft. of several dwellings, few complaints of odors have been made.

The sand filters are operated in rotation, the present maximum rate being about 300,000 gal. per acre daily. Except in winter the beds are scraped about every 16 days; when they are trenched, they receive attention about once in 10 days. With such care, no trouble has been experienced from clogging and but little sand is lost during cleaning. During the early period of mismanagement of the plant, the surface had been spaded to a depth of 6 in., and a large amount of sand had become so clogged that about 16 per cent. of the original material had to be removed before the bed was made serviceable.

At the disinfecting plant, bleach is used at the rate of 165 lb. per 1,000,000 gal., one-half between 7 a.m. and 5 p.m. This is more than is needed for treating the intermittent filter effluent, but the excess is used because the drainage from the sludge beds also flows into the clear well where the chemical solution is applied. When the sludge beds are not draining, there is disinfection of the brook water below the sewer outlet. Tests for B. coli during a period of 22 months showed the absence of this type of bacteria in all 1-cc. and smaller samples of the sterilized effluent, and its presence in but 14 per cent. of the 10-cc. samples. B. coli were present in the brook water above the outlet in all 10-cc. samples, 95 per cent. of the 1-cc., 60 per cent. of the 0.1-cc., and 5 per cent. of the 0.01-cc.

The operation of this plant was investigated by the New York State Department of Health in September, 1914, and the following notes on the results are from the report of Chief Engineer Theodore Horton. The leading analytical results are given in Table 184.

The raw sewage was graded in the report as about four and one-half times stronger than the average day flow of domestic sewage from a small city, which was attributed mainly to the laundry wastes in it. Its freshness was indicated by the 36 per cent. saturation with oxygen.

The tank treatment produced an effluent graded as about twice as strong as domestic sewage. Septic action was considered probable on account of the absence of dissolved oxygen.

The low result of the oxygen-consumed test of the settled effluent from the trickling filters is attributed to the rather long detention period in the settling tank. The effluent contained 5.3 parts of nitrates, while the effluent of the sand filters contained 22.2 parts.

TABLE 184.—RESULTS OF ANALYSES OF EFFLUENTS OF DIFFERENT STAGES OF TREATMENT AT THE HEBREW SHELTERING GUARDIAN ORPHANAGE

Stage	Oxygen consumed, 10 minutes boiling, parts per 1,000,000	Dissolved oxygen		Putrescibility, days	Bacteria, 20° agar
		Parts per 1,000,000	Percentage saturation		
Raw sewage	470	2.3	36	....	....
Tank effluent	220	0.0	0	....	7,000,000
Trickling filter effluent after sedimentation	39	1.6	18	2.6	2,400,000
Sand filter effluent	7.2	4.35	49	20+	16,000
Final effluent	6.5	7.8	87	.....[1].	20
Stream above outlet	1.9	5.1	58	20+	200
Stream below outlet	....	....	....	20+	5

[1] Test unreliable on account of action of bleach on methylene blue.

Quantity of Sewage from Institutions.—The amount of sewage for which provision must be made in these institutional plants varies greatly with the extent of the laundry and bathing activity, the admission of roof water to the sewers and the amount of ground-water infiltration. In the case of the treatment plant of the Boys Industrial School at Lancaster, O., the State Board of Health reported in 1908 that the rate of flow between 10 p.m. and 5 a.m. was uniformly 37 gal. per capita, while the average for 24 hours was only 50 gal. The rates of flow given in Table 185, obtained by measurement except in three cases, indicate that institutional sewage is likely to have a considerable per capita volume.

TABLE 185.—QUANTITIES OF SEWAGE FROM INSTITUTIONS

Institution	Population	Gal. per capita daily
Alliance, O., Children's Home	175	80 measured
Bedford Station, N. Y., Montefiore Home	300	100 estimated
Collingwood, O., Lake Shore R. R. shops	1,525	131 measured
Dayton, O., Montgomery Co. Infirmary	350	21–29 measured
Gallipolis, O., County Infirmary	40	40 estimated
Gallipolis, O., State Hospital, main buildings	1,450	122 measured
Gallipolis, O., State Hospital, Cottage I	225	127–211 measured
Lake Kushaqua, N. Y., Stony Wold Sanatorium	125	120 estimated
Lancaster, Ohio, Boys' Industrial Home	1,150	50–55 measured
Mansfield, Ohio, State Reformatory	820	67–83 measured
Marietta, O., Washington C. Infirmary	80	31 measured
Masillon, O., State Hospital	1,600	137 measured
Morgan's, O., Institute for Feeble Minded	265	53 measured
Oakdale, Mass., County Truant School	50	60 measured
Pleasantville, N. Y., Hebrew Children's Home	1,100	40–100 measured
Sandusky, Ohio, Soldiers' and Sailors' Home	1,125	151 measured
Southboro, Mass., St. Mark's School	216	140 measured
Toledo, O., State Hospital	2,000	150 measured

APPENDIX

CONVERSION TABLES

1,000,000 U. S. gal. per day = 1.547 second-feet.
1,000,000 U. S. gal. per day = 61.9 California statutory miner's inches.
1,000,000 U. S. gal. per day = 77.4 Southern California miner's inches.
1,000,000 U. S. gal. per day = 3.07 acre-feet.
1,000,000 U. S. gal. per acre = 206.6 U. S. gal. per cu. yd. of filter 3 ft. deep.
1 Imp. gal. per day per square yard = 5809 U. S. gal. per day per acre.

1 second-foot = 40 California statutory miner's inches.
1 second-foot = 50 Southern California miner's inches.
1 second-foot = 7.48 U. S. gal. per second.
1 second-foot = 448.8 U. S. gal. per minute.
1 second-foot = 646,317 U. S. gal. per day.
1 second-foot = about 1 acre-inch per hour.

1 acre-foot = 325,850 U. S. gallons.
1 acre-inch = 134 cu. yd.

1 U. S. gal.	= 0.8331 Imp. gal.	1 Imp. gal.	= 1.2003 U. S. gal.
1 U. S. gal.	= 3.78543 liters.	1 liter	= 0.26417 U. S. gal.
1 U. S. gal.	= 231 cu. in.	1 Imp. gal.	= 277.274 cu. in.

1 kilometer	= 0.62137 mile.	1 mile	= 1.60935 kilometers.
1 square meter	= 10.764 sq. ft.	1 square foot	= 0.0929 sq. meter.
1 square meter	= 1.1960 sq. yd.	1 square yard	= 0.836 sq. meter.
1 square kilometer	= 0.3861 sq. mile.	1 square mile	= 2.59 sq. km.
1 square millimeter	= 0.00155 sq. in.	1 square inch	= 645.2 sq. mm.
1 square centimeter	= 0.1550 sq. in.	1 square inch	= 6.452 sq. cm.
1 cubic millimeter	= 0.000061 cu. in.	1 cubic inch	= 16,387 cu. mm.
1 cubic centimeter	= 0.0610 cu. in.	1 cubic inch	= 16.387 cu. cm.
1 cubic centimeter	= 0.0338 fl. oz.	1 fluid ounce	= 29.57 cu. cm.
1 hectare	= 2.471 acres.	1 acre	= 0.4047 hectare.
1 gram	= 15.4324 grains.	1 grain	= 0.0648 gram.
1 kilogram	= 2.205 pounds.	1 pound	= 0.4536 kilogram.

1 English ton = 2240 pounds.
1 English hundredweight = 112 pounds.

Table A.—Fahrenheit into Centigrade Degrees

F.°	0	1	2	3	4	5	6	7	8	9
210	98.9	99.4	100.0	100.6	101.1	101.7	102.2	102.8	103.3	103.9
200	93.3	93.9	94.4	95.9	95.6	96.1	96.7	97.2	97.8	98.3
190	87.8	88.3	88.9	89.4	90.0	90.6	91.1	91.7	92.2	92.8
180	82.2	82.8	83.3	83.9	84.4	85.0	85.6	86.1	86.7	87.2
170	76.7	77.2	77.8	78.3	78.9	79.4	80.0	80.6	81.1	81.7
160	71.1	71.7	72.2	72.8	73.3	73.9	74.4	75.0	75.6	76.1
150	65.6	66.1	66.7	67.2	67.8	68.3	68.9	69.4	70.0	70.6
140	60.0	60.6	61.1	61.7	62.2	62.8	63.3	63.9	64.4	65.0
130	54.4	55.0	55.6	56.1	56.7	57.2	57.8	58.3	58.9	59.4
120	48.9	49.4	50.0	50.6	51.1	51.7	52.2	52.8	53.3	53.9
110	43.3	43.9	44.4	45.0	45.6	46.1	46.7	47.2	47.8	48.3
100	37.8	38.3	38.9	39.4	40.0	40.6	41.1	41.7	42.2	42.8
90	32.2	32.8	33.3	33.9	34.4	35.0	35.6	36.1	36.7	37.2
80	26.7	27.2	27.8	28.3	28.9	29.4	30.0	30.6	31.1	31.7
70	21.1	21.7	22.2	22.8	23.3	23.9	24.4	25.0	25.6	26.1
60	15.6	16.1	16.7	17.2	17.8	18.3	18.9	19.4	20.0	20.6
50	10.0	10.6	11.1	11.7	12.2	12.8	13.3	13.9	14.4	15.0
40	4.4	5.0	5.6	6.1	6.7	7.2	7.8	8.3	8.9	9.4
30	−1.1	−0.6	0.0	0.6	1.1	1.7	2.2	2.8	3.3	3.9
20	−6.7	−6.1	−5.6	−5.0	−4.4	−3.9	−3.3	−2.8	−2.2	−1.7
10	−12.2	−11.7	−11.1	−10.6	−10.0	−9.4	−8.9	−8.3	−7.8	−7.2
0	−17.8	−17.2	−16.7	−16.1	−15.6	−15.0	−14.4	−13.9	−13.3	−12.8

Table B.—Centigrade into Fahrenheit Degrees

C.°	0	1	2	3	4	5	6	7	8	9
100	212.0	213.8	215.6	217.4	219.2	221.0	222.8	224.6	226.4	228.2
90	194.0	195.8	197.6	199.4	201.2	203.0	204.8	206.6	208.4	210.2
80	176.0	177.8	179.6	181.4	183.2	185.0	186.8	188.6	190.4	192.2
70	158.0	159.8	161.6	163.4	165.2	167.0	168.8	170.6	172.4	174.2
60	140.0	141.8	143.6	145.4	147.2	149.0	150.8	152.6	154.4	156.2
50	122.0	123.8	125.6	127.4	129.2	131.0	132.8	134.6	136.4	138.2
40	104.0	105.8	107.6	109.4	111.2	113.0	114.8	116.6	118.4	120.2
30	86.0	87.8	89.6	91.4	93.2	95.0	96.8	98.6	100.4	102.2
20	68.0	69.8	71.6	73.4	75.2	77.0	78.8	80.6	82.4	84.2
10	50.0	51.8	53.6	55.4	57.2	59.0	60.8	62.6	64.4	66.2
0	32.0	33.8	35.6	37.4	39.2	41.0	42.8	44.6	46.4	48.2

Table C.—Grains per Imperial Gallon into Parts per Million

Gr.	0	1	2	3	4	5	6	7	8	9
0	0.0	14.3	28.6	42.8	57.1	71.4	85.7	100.0	114.3	128.6
10	142.9	157.1	171.4	185.7	200.0	214.3	228.6	242.9	257.2	271.4
20	285.7	300.0	314.3	328.6	342.9	357.1	371.4	385.7	400.0	414.3
30	428.6	442.9	457.1	471.4	485.7	500.0	514.3	528.6	542.9	557.1
40	571.4	585.7	600.0	614.3	628.6	642.9	657.2	671.4	685.7	700.0
50	714.3	728.6	742.9	757.1	761.4	785.7	800.0	814.3	828.6	832.9
60	857.2	871.4	885.7	900.0	914.3	928.6	942.9	957.2	971.4	985.7
70	1000.0	1044.3	1028.6	1042.8	1057.1	1071.4	1085.7	1100.0	1114.3	1128.6
80	1142.9	1157.1	1171.4	1185.7	1200.0	1214.3	1228.6	1242.9	1257.2	1271.4
90	1285.7	1300.0	1314.3	1328.6	1342.9	1357.1	1371.4	1385.7	1400.0	1414.3

Based on a weight of 10 lb. per Imperial gallon for water at 62°F.; equal to 70,000 grains.

Table D.—Grains per United States Gallon into Parts per Million

Gr.	0	1	2	3	4	5	6	7	8	9
0	0.0	17.1	34.3	51.4	68.6	85.7	102.8	120.0	137.1	154.2
10	171.4	188.5	205.7	222.8	239.9	257.1	274.2	291.4	308.5	325.6
20	342.8	359.9	377.1	394.2	411.3	428.5	445.6	462.7	479.9	497.0
30	514.2	531.3	548.4	565.6	582.7	599.8	617.0	634.1	651.3	668.4
40	685.5	702.0	719.8	737.0	754.1	771.2	788.4	805.5	822.6	839.8
50	856.9	874.1	891.2	908.3	925.5	942.6	959.8	976.9	994.0	1011.2
60	1028.3	1045.4	1062.6	1079.7	1096.9	1114.0	1131.1	1148.3	1165.4	1182.5
70	1199.7	1216.8	1234.0	1251.1	1268.3	1285.4	1302.5	1319.7	1336.8	1353.9
80	1371.1	1388.2	1405.4	1422.5	1439.7	1456.8	1473.9	1491.1	1508.2	1525.3
90	1542.4	1559.5	1576.7	1593.8	1611.0	1628.1	1645.2	1662.4	1679.5	1696.6

Water is assumed to be at 62°F. and to weigh 62.354 lb. per cubic foot at that temperature; or 1 gal. to weigh 58,349 grains.

Table E.—Parts per Million Parts into Pounds per Million U. S. Gallons

Parts	0	1	2	3	4	5	6	7	8	9
0	0.0	8.3	16.7	25.0	33.3	41.7	50.0	58.4	66.7	75.0
10	83.4	91.7	100.1	108.4	116.7	125.1	133.4	141.8	150.1	158.4
20	166.7	175.0	183.4	191.7	200.0	208.4	216.7	225.1	233.4	241.7
30	250.0	258.3	266.7	275.0	283.3	291.7	300.0	308.4	316.7	325.0
40	333.4	341.7	350.1	358.4	366.7	375.1	383.4	391.8	400.1	408.4
50	416.7	425.0	433.4	441.7	450.0	458.4	466.7	475.1	483.4	491.7
60	500.0	508.3	516.7	525.0	533.3	541.7	550.0	558.4	566.7	575.0
70	583.4	591.7	600.1	608.4	616.7	625.1	633.4	641.8	650.1	658.4
80	666.8	675.1	683.5	691.8	700.1	708.5	716.8	725.2	733.5	741.8
90	750.2	758.5	766.9	775.2	783.5	791.9	800.2	808.6	816.9	825.2

Based on weight of water at 62°F. = 8.33552 lb. per gallon.

TABLE F.—POUNDS PER MILLION U.S. GALLONS INTO PARTS PER MILLION

Pounds	0	10	20	30	40	50	60	70	80	90
0	0.0	1.2	2.4	3.6	4.8	6.0	7.2	8.4	9.6	10.8
100	12.0	13.2	14.4	15.6	16.8	18.0	19.2	20.4	21.6	22.8
200	24.0	25.2	26.4	27.6	28.8	30.0	31.2	32.4	33.6	34.8
300	36.0	37.2	38.4	39.6	40.8	42.0	43.2	44.4	45.6	46.8
400	48.0	49.2	50.4	51.6	52.8	54.0	55.2	56.4	57.6	58.8
500	60.0	61.2	62.4	63.6	64.8	66.0	67.2	68.4	69.6	70.8
600	72.0	73.2	74.4	75.6	76.8	78.0	79.2	80.4	81.6	82.8
700	84.0	85.2	86.4	87.6	88.8	90.0	91.2	92.4	93.6	94.8
800	96.0	97.2	98.4	99.6	100.8	102.0	103.2	104.4	105.6	106.8
900	108.0	109.2	110.4	111.6	112.8	114.0	115.2	116.4	117.6	118.8

TABLE G.—POUNDS PER MILLION U. S. GALLONS INTO GRAINS PER UNITED STATES GALLON

Pounds	0	10	20	30	40	50	60	70	80	90
0	0.00	0.07	0.14	0.21	0.28	0.35	0.42	0.49	0.56	0.63
100	0.70	0.77	0.84	0.91	0.98	1.05	1.12	1.19	1.26	1.33
200	1.40	1.47	1.54	1.61	1.68	1.75	1.82	1.89	1.96	2.03
300	2.10	2.17	2.24	2.31	2.38	2.45	2.52	2.59	2.66	2.73
400	2.80	2.87	2.94	3.01	3.08	3.15	3.22	3.29	3.36	3.43
500	3.50	3.57	3.64	3.71	3.78	3.85	3.92	3.99	4.06	4.13
600	4.20	4.27	4.34	4.41	4.48	4.55	4.62	4.69	4.76	4.83
700	4.90	4.97	5.04	5.11	5.18	5.25	5.32	5.39	5.46	5.53
800	5.60	5.67	5.74	5.81	5.88	5.95	6.02	6.09	6.16	6.23
900	6.30	6.37	6.44	6.51	6.58	6.65	6.72	6.79	6.86	6.93

TABLE H.—MILLIMETERS INTO INCHES

Mm.	0	1	2	3	4	5	6	7	8	9
0	......	0.039	0.079	0.118	0.157	0.197	0.236	0.276	0.315	0.354
10	0.394	0.433	0.472	0.512	0.551	0.591	0.630	0.669	0.709	0.748
20	0.787	0.827	0.866	0.906	0.945	0.984	1.024	1.063	1.102	1.142
30	1.181	1.220	1.260	1.299	1.339	1.378	1.417	1.457	1.496	1.535
40	1.575	1.614	1.654	1.693	1.732	1.772	1.811	1.850	1.890	1.929
50	1.969	2.008	2.047	2.087	2.126	2.165	2.205	2.244	2.283	2.323
60	2.362	2.402	2.441	2.480	2.520	2.559	2.598	2.638	2.677	2.717
70	2.756	2.795	2.835	2.874	2.913	2.953	2.992	3.031	3.071	3.110
80	3.150	3.189	3.228	3.268	3.307	3.346	3.386	3.425	3.465	3.504
90	3.543	3.583	3.622	3.661	3.701	3.740	3.780	3.819	3.858	3.898

This table can also be used to convert centimeters into inches, if it is kept in mind that 1 cm. = 10 mm.

Table I.—Meters into Feet

M.	0	1	2	3	4	5	6	7	8	9
0	0.0	3.28	6.56	9.84	13.12	16.40	19.69	22.97	26.25	29.53
10	32.81	36.09	39.37	42.65	45.93	49.21	52.49	55.77	59.06	62.34
20	65.62	68.90	72.18	75.46	78.74	82.02	85.30	88.58	91.86	95.14
30	98.43	101.71	104.99	108.27	111.55	114.83	118.11	121.39	124.67	127.95
40	131.23	134.51	137.80	141.08	144.36	147.64	150.92	154.20	157.48	160.76
50	164.04	167.32	170.60	173.88	177.17	180.45	183.73	187.01	190.29	193.57
60	196.85	200.13	203.41	206.69	209.97	213.25	216.54	219.82	223.10	226.38
70	229.66	232.94	236.22	239.50	242.78	246.06	249.34	252.62	255.91	259.19
80	262.47	265.75	269.03	272.31	275.59	278.87	282.15	285.43	288.71	291.99
90	295.28	298.56	301.84	305.12	308.40	311.68	314.96	318.24	321.52	324.80

Table J.—Millimeters per Second into Inches per Minute

Mm.	0	1	2	3	4	5	6	7	8	9
0	0.0	2.4	4.7	7.1	9.4	11.8	14.2	16.6	18.9	21.3
10	23.6	26.0	28.3	30.7	33.0	35.4	37.8	40.2	42.5	44.9
20	47.2	49.6	51.9	54.3	56.6	59.0	61.4	63.8	66.1	68.5
30	70.9	73.3	75.6	78.0	80.3	82.7	85.1	87.5	89.8	92.2
40	94.5	96.9	99.2	101.6	103.9	106.3	108.7	111.1	113.4	115.8
50	118.1	120.5	122.8	125.2	127.5	129.9	132.3	134.7	137.0	139.4
60	141.7	144.1	146.4	148.8	151.1	153.5	155.9	158.3	160.6	163.0
70	165.4	167.8	170.1	172.5	174.8	177.2	179.6	182.0	184.3	186.7
80	189.0	191.4	193.7	196.1	198.4	200.8	203.2	205.6	207.9	210.3
90	212.6	215.0	217.3	219.7	222.0	224.4	226.8	229.2	231.5	233.9

Table K.—Millimeters per Second into Feet per Hour

Mm.	0	1	2	3	4	5	6	7	8	9
0	0.0	11.8	23.6	35.4	47.2	59.1	70.9	82.7	94.5	106.3
10	118.1	129.9	141.7	153.5	165.3	177.2	189.0	200.8	212.6	224.4
20	236.2	248.0	259.8	271.6	283.4	295.3	307.1	318.9	330.7	342.5
30	354.3	366.1	377.9	389.7	401.5	413.4	425.2	437.0	448.8	460.6
40	472.4	484.2	496.0	507.8	519.6	531.5	543.3	555.1	566.9	578.7
50	590.6	602.4	614.2	626.0	637.8	649.7	661.5	673.3	685.1	696.9
60	708.7	720.5	732.3	744.1	755.9	767.8	779.6	791.4	802.2	815.0
70	826.8	838.6	850.4	862.2	874.0	885.9	897.7	909.5	921.3	933.1
80	944.9	956.7	968.5	980.3	992.1	1004.0	1015.8	1027.6	1039.4	1051.2
90	1062.9	1074.7	1086.5	1098.3	1110.1	1122.0	1133.8	1145.6	1157.4	1169.2

TABLE L.—CUBIC METERS INTO CUBIC YARDS

Cu. m.	0	1	2	3	4	5	6	7	8	9
0	......	1.31	2.62	3.92	5.23	6.54	7.85	9.16	10.46	11.77
10	13.08	14.39	15.70	17.00	18.31	19.62	20.93	22.24	23.54	24.85
20	26.16	27.47	28.77	30.08	31.39	32.70	34.01	35.31	36.62	37.93
30	39.24	40.55	41.85	43.16	44.47	45.78	47.09	48.39	49.70	51.01
40	52.32	53.63	54.93	56.24	57.55	58.86	60.17	61.47	62.78	64.09
50	65.40	66.71	68.01	69.32	70.63	71.94	73.24	74.55	75.86	77.17
60	78.48	79.78	81.09	82.40	83.71	85.02	86.32	87.63	88.94	90.25
70	91.56	92.86	94.17	95.48	96.79	98.10	99.40	100.71	102.02	103.33
80	104.64	105.94	107.25	108.56	109.87	111.18	112.48	113.79	115.10	116.41
90	117.71	119.02	120.33	121.64	122.95	124.25	125.56	126.87	128.18	129.49

TABLE M.—CUBIC METERS INTO CUBIC FEET

Cu. m.	0	1	2	3	4	5	6	7	8	9
0	......	35.3	70.6	105.9	141.3	176.6	211.9	247.2	282.5	317.8
10	353.1	388.5	423.8	459.1	494.4	529.7	565.0	600.3	635.7	671.0
20	706.3	741.6	776.9	812.2	847.5	882.9	918.2	953.5	988.8	1024.1
30	1059.4	1094.7	1130.1	1165.4	1200.7	1236.0	1271.3	1306.6	1341.9	1377.3
40	1412.6	1447.9	1483.2	1518.5	1553.8	1589.2	1624.5	1659.8	1695.1	1730.4
50	1765.7	1801.0	1836.4	1871.7	1907.0	1942.3	1977.6	2012.9	2048.2	2083.6
60	2118.9	2154.2	2189.5	2224.8	2260.1	2295.4	2230.8	2366.1	2401.4	2436.7
70	2472.0	2507.3	2542.6	2578.0	2613.3	2648.6	2683.9	2719.2	2754.5	2789.8
80	2825.2	2860.5	2895.8	2931.1	2966.4	3001.7	3037.0	3072.4	3107.7	3143.0
90	3178.3	3213.6	3248.9	3284.2	3319.6	3354.9	3390.2	3425.5	3460.8	3496.1

TABLE N.—LITERS PER SECOND INTO MILLIONS OF U. S. GALLONS PER DAY

Liters	0	1	2	3	4	5	6	7	8	9
0	0.0	0.023	0.046	0.068	0.090	0.113	0.136	0.159	0.182	0.204
10	0.228	0.251	0.274	0.296	0.318	0.341	0.364	0.387	0.410	0.432
20	0.456	0.479	0.502	0.524	0.546	0.569	0.592	0.615	0.638	0.660
30	0.675	0.698	0.721	0.743	0.765	0.788	0.811	0.834	0.857	0.879
40	0.903	0.926	0.949	0.971	0.993	1.016	1.039	1.062	1.085	1.107
50	1.131	1.154	1.177	1.199	1.221	1.244	1.267	1.290	1.313	1.335
60	1.359	1.382	1.405	1.427	1.449	1.472	1.495	1.518	1.541	1.563
70	1.588	1.611	1.634	1.656	1.678	1.701	1.724	1.747	1.770	1.792
80	1.816	1.839	1.862	1.884	1.906	1.929	1.952	1.975	1.998	2.020
90	2.044	2.067	2.090	2.112	2.134	2.157	2.180	2.203	2.226	2.248

TABLE O.—U. S. GALLONS PER MINUTE INTO CUBIC FEET PER SECOND

Gal.	0	1	2	3	4	5	6	7	8	9
0	0.0	0.002	0.005	0.007	0.009	0.011	0.013	0.016	0.018	0.020
10	0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.038	0.040	0.042
20	0.045	0.047	0.050	0.052	0.054	0.056	0.058	0.061	0.063	0.065
30	0.067	0.069	0.072	0.074	0.076	0.078	0.080	0.083	0.085	0.087
40	0.089	0.091	0.094	0.096	0.098	0.100	0.102	0.105	0.107	0.109
50	0.111	0.113	0.116	0.118	0.120	0.122	0.124	0.127	0.129	0.131
60	0.134	0.136	0.139	0.141	0.143	0.145	0.147	0.150	0.152	0.154
70	0.156	0.158	0.161	0.163	0.165	0.167	0.169	0.172	0.174	0.176
80	0.178	0.180	0.183	0.185	0.187	0.189	0.191	0.194	0.196	0.198
90	0.200	0.202	0.205	0.207	0.209	0.211	0.213	0.216	0.218	0.220

TABLE P.—GALLONS PER MINUTE INTO MILLIONS OF GALLONS PER DAY

Gal./min.	0	10	20	30	40	50	60	70	80	90
0	0.0	0.014	0.029	0.043	0.058	0.072	0.087	0.101	0.116	0.130
100	0.144	0.158	0.173	0.187	0.202	0.216	0.231	0.245	0.260	0.274
200	0.289	0.303	0.318	0.332	0.347	0.361	0.376	0.390	0.405	0.419
300	0.433	0.447	0.462	0.476	0.491	0.505	0.520	0.534	0.549	0.563
400	0.578	0.592	0.607	0.621	0.636	0.650	0.665	0.679	0.694	0.708
500	0.722	0.736	0.751	0.765	0.780	0.794	0.809	0.823	0.838	0.852
600	0.866	0.880	0.895	0.909	0.924	0.938	0.953	0.967	0.982	0.996
700	1.011	1.025	1.040	1.054	1.069	1.083	1.098	1.112	1.127	1.141
800	1.155	1.169	1.184	1.198	1.213	1.227	1.242	1.256	1.271	1.285
900	1.300	1.314	1.329	1.343	1.358	1.372	1.387	1.401	1.416	1.430

TABLE Q.—CUBIC METERS PER SECOND INTO MILLIONS OF U. S. GALLONS PER DAY

C.m./Sec.	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.0	2.282	4.565	6.747	9.030	11.312	13.594	15.877	18.159	20.442
1	22.824	25.106	27.389	29.571	31.854	34.136	36.418	38.701	40.983	43.266
2	45.648	47.930	50.213	52.395	54.678	56.960	59.242	61.525	63.807	66.090
3	67.472	69.754	72.037	74.219	76.502	78.784	81.066	83.349	85.631	87.914
4	90.296	92.578	94.861	97.043	99.326	101.608	103.890	106.173	108.455	110.738
5	113.120	115.402	117.685	119.867	122.150	124.432	126.714	128.997	131.279	133.562
6	135.944	138.226	140.509	142.691	144.974	147.256	149.538	151.821	154.103	156.386
7	158.768	161.050	163.333	165.515	167.798	170.080	172.362	174.645	176.927	179.210
8	181.592	183.874	186.157	188.339	190.622	192.904	195.186	197.469	199.751	202.034
9	204.416	206.698	208.981	211.163	213.446	215.728	218.010	220.293	222.575	224.858

TABLE R.—CUBIC FEET PER SECOND INTO MILLIONS OF U. S. GALLONS PER DAY

Cu. ft./Sec.	0	1	2	3	4	5	6	7	8	9
0	0.0	0.646	1.293	1.939	2.585	3.232	3.878	4.524	5.171	5.817
10	6.463	7.109	7.756	8.402	9.048	9.695	10.341	10.987	11.634	12.280
20	12.926	13.572	14.219	14.865	15.511	16.158	16.804	17.450	18.097	18.743
30	19.389	20.035	20.682	21.328	21.974	22.621	23.267	23.913	24.560	25.206
40	25.852	26.498	27.145	27.791	28.437	29.084	29.730	30.376	31.023	31.669
50	32.316	32.962	33.609	34.255	34.901	35.548	36.194	36.840	37.487	38.133
60	38.779	39.425	40.072	40.718	41.364	42.011	42.657	43.303	43.950	44.596
70	45.242	45.888	46.535	47.181	47.827	48.474	49.120	49.766	50.413	51.059
80	51.705	52.351	52.998	53.644	54.290	54.937	55.583	56.229	56.876	57.522
90	58.168	58.814	59.461	60.107	60.753	61.400	62.046	62.692	63.339	63.985

TABLE S.—LITERS IN TO GALLONS

L.	0	1	2	3	4	5	6	7	8	9
0	0.0	0.264	0.528	0.793	1.057	1.321	1.585	1.849	2.113	2.378
10	2.642	2.906	3.170	3.434	3.698	3.963	4.227	4.491	4.755	5.019
20	5.283	5.478	5.811	6.076	6.340	6.604	6.868	7.133	7.397	7.661
30	7.925	8.189	8.453	8.718	8.982	9.246	9.510	9.774	10.038	10.303
40	10.567	10.831	11.095	11.359	11.624	11.888	12.152	12.416	12.680	12.944
50	13.209	13.473	13.737	14.001	14.265	14.529	14.794	18.058	15.322	15.586
60	15.850	16.114	16.379	16.643	16.907	17.171	17.435	17.699	17.964	18.228
70	18.492	18.756	19.020	19.284	19.549	19.813	20.077	20.341	20.605	20.869
80	21.134	21.398	21.662	21.926	22.190	22.454	22.719	22.983	23.247	23.511
90	23.775	24.040	24.304	24.568	24.832	25.096	25.360	25.625	25.889	26.153

TABLE T.—KILOGRAMS IN TO POUNDS AVOIRDUPOIS

Kg	0	1	2	3	4	5	6	7	8	9
0	0.0	2.205	4.409	6.614	8.819	11.023	13.228	15.432	17.637	19.842
10	22.046	24.251	26.456	28.660	30.865	33.069	35.274	37.479	39.683	41.888
20	44.092	46.297	48.502	50.706	52.911	55.116	57.320	59.525	61.729	63.934
30	66.139	68.343	70.548	72.753	74.957	77.162	79.366	81.571	83.776	85.980
40	88.185	90.390	92.594	94.799	97.003	99.208	101.413	103.617	105.822	108.027
50	110.231	112.436	114.640	116.845	119.050	121.254	123.459	125.664	127.868	130.073
60	132.277	134.482	136.687	138.891	141.096	143.301	145.505	147.710	149.914	152.119
70	154.324	156.528	158.733	160.937	163.142	165.347	167.551	169.756	171.961	174.165
80	176.370	178.574	180.779	182.984	185.188	187.393	189.598	191.802	194.007	196.201
90	198.416	200.621	202.825	205.030	207.235	209.439	211.644	213.848	216.053	218.258

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